EFFECTS OF SEABED INTERACTION ON STEEL CATENARY RISERS

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ABSTRACT

Steel catenary risers are an enabling technology for deepwater oil and gas production. A steel catenary riser consists of a steel pipeline suspended between the vessel and the seabed forming a catenary shape. Tools to analyse and design steel catenary risers show that the point where the steel catenary riser first touches the seabed, termed the touchdown point, has the highest stress and the greatest fatigue damage. Current understanding of pipe/soil interaction is limited and consequently there is concern within the industry regarding the conservatism of the analysis. In particular, the implications of pipe/soil interaction for maximum stress and fatigue damage at the touchdown point are significant.

To address these concerns, research has been conducted into the following areas:

- Steel catenary riser trenches – using video survey data from installed steel catenary risers to determine the shape of seabed trenches. A steel catenary riser trench profile has been developed for use in finite element analysis.
- Pipe/soil suction force – i.e. the bond that forms between the riser pipe and a clay seabed. Experiments have been conducted and a pipe/soil suction model developed for use in steel catenary riser analysis.
- Pipe/soil stiffness – test data from the CARISIMA and STRIDE JIPs has been examined and a series of soil stiffness models for static penetration, small and large displacements, and cyclic loading have been developed for use in finite element analysis programs.
- Closed form and finite element models of steel catenary risers were constructed to determine the effect of the soil on stress and fatigue damage at the touchdown point.

A finite element model of a representative steel catenary riser has been created and analysed using the seabed interaction models developed. The results show that the seabed trench, pipe/soil suction and soil stiffness have little effect on extreme stress in the steel catenary riser during normal operating conditions. However, pipe/soil suction is shown to have a large effect during slow drift motions where the stress in the riser at the touchdown point could double. The results from a closed form seabed model and finite element analysis show that the fatigue life of a steel catenary riser is sensitive to soil stiffness. If the soil stiffness used to model the seabed is too high the fatigue life may be underestimated; conversely, if the soil stiffness is too low the fatigue life may be over estimated.
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<td>characteristic area (m)</td>
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<tr>
<td>A</td>
<td>horizontal projection of contact area (m)</td>
</tr>
<tr>
<td>$A_S$</td>
<td>cross-sectional area of pipe ($m^2$)</td>
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<td>B</td>
<td>bearing width of foundation on elastic surface (m)</td>
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<td>C</td>
<td>number of cycles (-)</td>
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<td>$c_V$</td>
<td>coefficient of consolidation ($m^2$/year)</td>
</tr>
<tr>
<td>$D, D_0$</td>
<td>pipe outer diameter (m)</td>
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<tr>
<td>$D_E$</td>
<td>degradation factor (-)</td>
</tr>
<tr>
<td>$D_{fR}$</td>
<td>damage factor ratio (-)</td>
</tr>
<tr>
<td>$D_i$</td>
<td>pipe inner diameter (m)</td>
</tr>
<tr>
<td>$D_{fR}$</td>
<td>fatigue damage rate (1/years)</td>
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<tr>
<td>$E$</td>
<td>Young's modulus ($N/m^2$)</td>
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<tr>
<td>$E_i$</td>
<td>Young's modulus of soil for the first cycle ($N/m^2$)</td>
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<tr>
<td>$E_n$</td>
<td>Young’s modulus of soil after n cycles ($N/m^2$)</td>
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<tr>
<td>$e_s$</td>
<td>void ratio (-)</td>
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<tr>
<td>F</td>
<td>force (N)</td>
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<td>$F_C$</td>
<td>consolidation load (N)</td>
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<tr>
<td>$F_{\text{max}}$</td>
<td>maximum force (N)</td>
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<tr>
<td>$G_s$</td>
<td>specific gravity of soil (-)</td>
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<tr>
<td>g</td>
<td>gravity, 9.81 m/s$^2$ (m/s$^2$)</td>
</tr>
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<td>H</td>
<td>depth of trench (m)</td>
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<td>$H$</td>
<td>horizontal load of catenary (N)</td>
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<td>h</td>
<td>depth of foundation (m)</td>
</tr>
<tr>
<td>I</td>
<td>second moment of area ($m^4$)</td>
</tr>
<tr>
<td>$I_p$</td>
<td>plasticity index (%)</td>
</tr>
<tr>
<td>$I_{s}$</td>
<td>shape factor of a loaded area for elastic foundations (-)</td>
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<tr>
<td>J</td>
<td>dimensionless empirical constant for lateral pile/soil interaction (-)</td>
</tr>
<tr>
<td>K</td>
<td>soil stiffness ($N/m/m$)</td>
</tr>
<tr>
<td>$K_D$</td>
<td>dynamic pipe/soil stiffness ($N/m/m$)</td>
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<tr>
<td>$K_L$</td>
<td>large displacement pipe/soil stiffness ($N/m/m$)</td>
</tr>
<tr>
<td>$K_R$</td>
<td>non-dimensional soil rigidity parameter (-)</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>KS</td>
<td>static linear pipe/soil stiffness</td>
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<tr>
<td>KST</td>
<td>tangent static linear pipe/soil stiffness</td>
</tr>
<tr>
<td>k</td>
<td>structural stiffness</td>
</tr>
<tr>
<td>k</td>
<td>empirical constant for fatigue analysis</td>
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<tr>
<td>k</td>
<td>linear seabed stiffness for Winkler foundations</td>
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<tr>
<td>kD</td>
<td>empirical factor for pull-out velocity</td>
</tr>
<tr>
<td>kDC</td>
<td>empirical cyclic loading factor</td>
</tr>
<tr>
<td>kDT</td>
<td>empirical factor for consolidation time</td>
</tr>
<tr>
<td>kDTF</td>
<td>empirical derived constant for consolidation time</td>
</tr>
<tr>
<td>kDV</td>
<td>empirical pull-out velocity factor</td>
</tr>
<tr>
<td>kF</td>
<td>frequency factor</td>
</tr>
<tr>
<td>kH</td>
<td>hysteresis factor</td>
</tr>
<tr>
<td>k stiff</td>
<td>soil stiffness factor</td>
</tr>
<tr>
<td>kTDF</td>
<td>curvature in SCR at TDP</td>
</tr>
<tr>
<td>kV</td>
<td>empirical pull-out velocity factor</td>
</tr>
<tr>
<td>L</td>
<td>pipe length</td>
</tr>
<tr>
<td>L</td>
<td>fatigue life</td>
</tr>
<tr>
<td>MO</td>
<td>applied moment</td>
</tr>
<tr>
<td>MPOS</td>
<td>maximum positive moment</td>
</tr>
<tr>
<td>MTDP</td>
<td>moment at the TDP</td>
</tr>
<tr>
<td>MX</td>
<td>moment at a distance x along the pipe</td>
</tr>
<tr>
<td>m</td>
<td>empirical constant for fatigue analysis</td>
</tr>
<tr>
<td>m</td>
<td>mass per unit length</td>
</tr>
<tr>
<td>ms</td>
<td>submerged mass per unit length</td>
</tr>
<tr>
<td>mV</td>
<td>coefficient of volume compressibility</td>
</tr>
<tr>
<td>N</td>
<td>number of cycles to failure for fatigue analysis</td>
</tr>
<tr>
<td>N</td>
<td>bearing capacity factor</td>
</tr>
<tr>
<td>NC</td>
<td>bearing capacity factor</td>
</tr>
<tr>
<td>Nch</td>
<td>horizontal bearing capacity factor</td>
</tr>
<tr>
<td>Ncv</td>
<td>vertical bearing capacity factor</td>
</tr>
<tr>
<td>NaN</td>
<td>number of cycles to failure for a given stress range</td>
</tr>
<tr>
<td>Nσ</td>
<td>number of stress ranges considered</td>
</tr>
<tr>
<td>nD</td>
<td>empirically derived constant for pull-out velocity</td>
</tr>
</tbody>
</table>
\( n_F \) empirically derived constant for pull-out velocity  
\( n_{TF} \) empirically derived constant for consolidation time  
\( n_{DTF} \) empirically derived constant for consolidation time  
\( P \) lateral force  
\( \text{PMAX} \) peak lateral force  
\( P_U \) ultimate lateral force  
\( P_r \) residual lateral force  
\( p_a \) atmospheric pressure  
\( p_{ult} \) ultimate lateral section pressure  
\( Q \) vertical force  
\( Q_S \) vertical upwards force  
\( Q_r \) residual vertical force  
\( Q_U \) ultimate vertical soil resistance force  
\( q \) pressure  
\( q_{ult} \) ultimate vertical section pressure  
\( R \) radius  
\( R_C \) concentrated reaction force at TDP  
\( R_P \) reaction along the pipe  
\( S \) length of riser from the TDP to a point on the riser  
\( S_R \) length of riser between the TDP and the vessel  
\( S_U \) undrained soil shear strength  
\( S_{UG} \) undrained soil shear strength gradient  
\( S_{U0} \) undrained soil shear strength at surface  
\( T \) tension  
\( T \) period  
\( T_{TOP} \) top tension of SCR  
\( T_V \) time factor  
\( T_W \) pipe wall thickness  
\( T_Z \) zero crossing period  
\( t \) time  
\( t_b \) breakout time  
\( t_0 \) reference time interval  
\( t_T \) total length of timetrace  
\( S_r \) degree of saturation
tu ultimate axial force (N)
U average degree of consolidation (-)
V pull-out velocity (m/s)
Vx shear force at a distance x along the pipe (N)
W point load (N)
wL liquid limit (%)
wP plastic limit (%)
X a soil parameter that varies between 0.85 for soft soils and 0.97 for stiff soils (-)
x axial in line with the pipe displacement (m)
xu ultimate axial displacement (m)
xtdp horizontal distance between the vessel and TDP (m)
y lateral displacement (m)
y distance from pipe centre line to extreme pipe fibre (m)
yy displacement at peak lateral force (m)
z vertical displacement (m)
z vertical axis for global riser models (m)
zA vertical distance between the seabed and the riser attachment point (m)
zD dynamic displacement (m)
zU ultimate vertical displacement (m)
zx displacement at a distance x along the pipe (m)
alpha tension beam on elastic foundation constant (-)
alpha angle between the catenary and the horizontal axis (°)
alphaTOP angle between the riser/vessel connection and the horizontal axis (°)
beta dimensionless parameter relating E to Su (-)
b activity parameter for passive soil resistance (-)
b tension beam on elastic foundation constant (-)
gamma shape factor (-)
ys submerged unit weight of soil (N/m³)
Delta displacement (m)
DeltaB breakout displacement (m)
Deltas change in stress (Pa)
\( \varepsilon \) strain (-)
\( \theta \) angle (°)
\( \theta \) top angle of SCR (°)
\( \theta_X \) angle at a distance \( x \) along the pipe (°)
\( \phi' \) angle of shearing resistance (shear strength parameter) (°)
\( \Lambda \) normalised mobilisation distance (-)
\( \lambda \) constant relating soil stiffness to bending stiffness (-)
\( \lambda_L \) flexural length parameter (m)
\( \mu \) coefficient of friction (-)
\( \mu_A \) coefficient of axial friction (-)
\( \mu_L \) coefficient of residual lateral load (-)
\( \nu \) Poisson’s ratio (-)
\( \rho \) density (kg/m³)
\( \rho_d \) bulk density (Mg/m³)
\( \rho_s \) density of steel, 7860kg/m³ (kg/m³)
\( \rho_w \) density of sea water, 1025kg/m³ (kg/m³)
\( \sigma_A \) axial stress (Pa)
\( \sigma_F \) stress range for fatigue analysis (Pa)
\( \sigma_R \) radial stress (Pa)
\( \sigma_{RMS} \) RMS stress (Pa)
\( \sigma_v' \) effective vertical stress (Pa)
\( \sigma_{VM} \) von Mises stress (Pa)
\( \sigma_\theta \) hoop stress (Pa)

APDL ANSYS Parametric Design Language
BWR Bottom Weighted Riser
CARISIMA Catenary Riser/Soil Interaction Model for Global Riser Analysis
DNV Det Norske Veritas
EDM Electromagnetic Distance Measurement
FE Finite Element
FEA Finite Element Analysis
<table>
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<tr>
<th>Abbreviation</th>
<th>Full Form</th>
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<tr>
<td>FPS</td>
<td>Floating Production System</td>
</tr>
<tr>
<td>GoM</td>
<td>Gulf of Mexico</td>
</tr>
<tr>
<td>GRP</td>
<td>Glass Reinforced Plastic</td>
</tr>
<tr>
<td>JIP</td>
<td>Joint Industry Project</td>
</tr>
<tr>
<td>LWR</td>
<td>Lazy Wave Riser</td>
</tr>
<tr>
<td>MSL</td>
<td>Mean Sea Level</td>
</tr>
<tr>
<td>OCR</td>
<td>Over Consolidation Ratio</td>
</tr>
<tr>
<td>PSP</td>
<td>Perforated Steel Planks</td>
</tr>
<tr>
<td>RAO</td>
<td>Response Amplitude Operator</td>
</tr>
<tr>
<td>ROV</td>
<td>Remotely Operated Vehicle</td>
</tr>
<tr>
<td>SCF</td>
<td>Stress Concentration Factor</td>
</tr>
<tr>
<td>SCR</td>
<td>Steel Catenary Riser</td>
</tr>
<tr>
<td>STRIDE</td>
<td>Steel Risers in Deepwater Environments</td>
</tr>
<tr>
<td>TDP</td>
<td>Touchdown Point</td>
</tr>
<tr>
<td>TDZ</td>
<td>Touchdown Zone</td>
</tr>
<tr>
<td>TLP</td>
<td>Tension Leg Platform</td>
</tr>
<tr>
<td>TTR</td>
<td>Top Tensioned Riser</td>
</tr>
<tr>
<td>VIV</td>
<td>Vortex Induced Vibration</td>
</tr>
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1.0 INTRODUCTION

1.1 Overview

Oil and gas are found in underground reservoirs where they have been produced over millions of years from the remains of dead plants and animals. Most of the oil and gas reserves on land or at shallow water depths have been tapped, as the technology to recover oil and gas has become available. As the oil and gas recovery technology develops, further exploration of increasingly deep offshore reservoirs becomes economically viable.

Shallow water oil exploration requires the use of either a fixed gravity structure or a floating production system, to enable access to the well via vertical pipelines, called risers. As the water depth increases, the choice of economic offshore platforms becomes limited, which affects the type of riser used. An enabling technology for deepwater oil and gas recovery is the steel catenary riser (SCR): a steel pipe suspended from the floating production system rising from the seabed in a catenary, as shown in Figure 1.1.

![Figure 1.1 - Steel Catenary Riser Arrangement](image-url)
Tools to analyse and design steel catenary risers are available which show that the point where the riser first touches the soil, termed the touchdown point (TDP), has the highest stresses and the greatest fatigue damage along the riser. However our understanding of pipe/soil interaction is limited, hence the industry has concerns regarding the levels of conservatism and the safety of SCR designs.

1.2 Steel Catenary Risers (SCR)

The concept of a SCR is fairly simple. It is a rigid pipeline suspended near vertically from a vessel, curving its way down to become a horizontal pipeline along the seabed. Since the SCR is essentially a pipeline, standard pipe sections can be used, which make SCRs economic to produce. The increasing number of new deepwater field developments (water depths of 500m plus) employing SCRs indicates this, Chaudhury (2001). SCRs have limited feasibility in shallow water, as the high bending moments and curvatures resulting from bending the pipeline into the catenary shape make them expensive for larger diameter pipes.

Initially SCRs were installed with tension leg platforms (TLP). This type of facility has a floating hull that is tethered to the seabed. Steel tendons hold the hull below its natural level of floatation, keeping the tendons in tension and the hull in place, Leffler et al (2003). TLPs are predominantly subject to surge and sway (horizontal) motions. As oil and gas reserves are found in deeper water alternative production vessels maybe selected, including semi-submersibles and ship-shaped floating production systems (FPS) that are subject to all six degrees of motion, including heave and roll, which generate more complex riser motions.

Calculating the static shape and forces on a SCR can be accomplished using standard catenary equations. More detailed analysis of risers can be conducted using non-linear finite element analysis programs. Most specialist state-of-the-art riser analysis codes use either rigid or linear elastic contact surfaces, to model the seabed and simulate vertical soil resistance to pipe penetration, horizontal friction resistance and axial friction resistance. A rigid surface generally gives a conservative result since it is unyielding, while the linear elastic surface is a better approximation of a seabed. Both rigid and flexible seabed approaches do not account for viscous, non-
linear or vertical uplift soil-structure interaction effects. While non-linear springs in combination with damping, sliding and control elements can be configured to model this, little knowledge is available to define the modelling parameters.

It has been shown by a number of authors including Vesic (1971) and Bostrom et al (1998) that little is known about the viscous, non-linear or vertical uplift soil/structure interaction effects on SCRs and consequently bring into question the conservatism of existing riser modelling techniques. The main focus of this thesis is the phenomena of vertical embedment and uplift resistance, (termed pipe/soil suction) and the effect of the seabed, including soil-structure interaction on SCR maximum stress and fatigue damage.

1.3 Experimental Studies

The experimental studies detailed within this thesis were conducted as part of the STRIDE JIP (Steel Risers in Deepwater Environments Joint Industry Project), (2H Offshore, 1998 – 2002). These include a series of full-scale experiments conducted on a test riser in a harbour, and 2D pipe/soil interaction experiments. The writer was an integral part of the STRIDE team that conducted the experimental studies.

The STRIDE JIP was also responsible for collecting ROV (remotely operated vehicle) surveys of SCR trenches from five existing developments. Analysis of the surveys is conducted in this thesis to gain a better understanding of SCR trenches and trenching mechanisms and to provide input to TDP modelling.

The full-scale experiments examined the three dimensional effect of pipe/soil interaction on a catenary riser. This allowed pipe/soil interaction effects to be observed along the riser length that would not have been obvious, or seen, in the small scale 2D experiments. The experiments were designed to examine the effects of pipe/soil suction, lateral soil resistance to 3D pipe movement, soil stiffness and trenching mechanisms.

The STRIDE small scale 2D experiments were conducted to examine pipe/soil interaction, in the form of force/displacement curves on a section of riser pipe, on a
marine clay under controlled conditions. These tests represented a section of riser pipe in a trench, but do not account for any effects caused by the catenary shape of the riser. Many tests were conducted and the effects of consolidation time, pipe weight, pull-out velocity and riser diameter examined.

In addition to the STRIDE tests, the raw pipe/soil interaction test data from the CARISIMA JIP (Catenary Riser/Soil Interaction Model for Global Riser Analysis Joint Industry Project), (Marintek, 2000a & b) was evaluated. The CARISIMA JIP data consisted of force and displacement timetraces from a series of 2D small-scale pipe/soil interaction tests, which included the pipe being pushed vertically into and pulled vertically and laterally out of the soil. These tests benefited from a much more sophisticated test rig compared to the STRIDE test rig, but were limited in the number of tests that could be conducted. The experimental studies used in this thesis are summarised in Table 1.1.

The data collected from the STRIDE and CARISIMA experimental studies was used to create pipe/soil interaction models for pipe/soil suction, static penetration of a pipe into a seabed and dynamic soil stiffness. The pipe/soil interaction models were verified using finite element analysis (FEA) and the data from the full-scale harbour test riser. These models were applied to a SCR using FEA and a closed form solution to determine the effect of seabed interaction on SCRs and identify critical parameters.
<table>
<thead>
<tr>
<th>Test Set Name</th>
<th>Date Conducted</th>
<th>Test Description</th>
</tr>
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<tr>
<td>ROV SCR Trench Surveys</td>
<td>1998 – 2002</td>
<td>ROV surveys of subsea SCR trenches</td>
</tr>
<tr>
<td>STRIDE Phase III JIP Harbour</td>
<td>January – December 2000</td>
<td>Full scale riser</td>
</tr>
<tr>
<td>Test Riser Experiments</td>
<td></td>
<td></td>
</tr>
<tr>
<td>STRIDE Phase IV JIP 2D Pipe/soil</td>
<td>March – May 2001</td>
<td>Small scale 2D</td>
</tr>
<tr>
<td>Interaction Tests</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CARISIMA I JIP 2D Pipe/soil</td>
<td>2001</td>
<td>Small scale 2D</td>
</tr>
<tr>
<td>Interaction Tests</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CARISIMA II JIP 2D Pipe/soil</td>
<td>January – June 2002</td>
<td>Small scale 2D</td>
</tr>
<tr>
<td>Interaction Tests</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1.4 Organisation of Thesis

The layout of this thesis is as follows. Chapter 2 contains a literature review covering SCRs, subsea soils and pipe/soil interaction. Chapter 3 contains analysis of ROV trench surveys. Chapters 4 and 5 cover the experimental work including the harbour test riser and the 2D pipe pull-out experiments examining vertical upward pipe/soil interaction. Chapter 6 shows how the pipe/soil suction model was derived from the test data. Chapter 7 details the development of a soil stiffness model using observations from the previous experiments and the analysis of the CARISIMA test data. A series of closed form pipe/seabed interaction models are derived and their predictions of SCR stress and fatigue life is examined in Chapter 8. Chapter 9 details the SCR finite element analysis using the derived pipe/soil suction and soil stiffness models and the effects on riser stress distribution and fatigue life. The conclusions are presented in Chapter 10 where recommendations for further work are also made.
2.0 LITERATURE REVIEW

2.1 Introduction

Steel catenary risers (SCRs) are a relatively new technology; the first SCRs were installed on Shell’s Auger platform, Gulf of Mexico, USA in 1994 (Serta et al, 1996). Consequently the oil and gas industry is still developing the SCR concept. It has been stated by Patel (1995) that there is not enough experimental validation of catenary riser (flexible and steel) and that more full scale and small scale experiments are required.

This section presents the theoretical considerations used in global SCR analysis. From here the focus moves to the touchdown zone (TDZ), then subsea soils and the mechanics of pipe/soil interaction. The individual vertical, lateral and axial models for pipe/soil interaction, followed by SCR subsea trench theories are then examined in detail.

2.2 Steel Catenary Risers (SCR)

2.2.1 Overview

SCRs are pipes that hang almost vertically from a floating production system (FPS) and curve down so they lie horizontally on the seabed. The point where the riser first touches the seabed is termed the touchdown point (TDP) and the area of dynamic pipe/soil interaction is the TDZ. Further on from the TDZ the riser becomes a static pipeline, lying on the surface of the seabed. Steel catenary risers may be described as consisting of three sections, Bridge et al (2003), as shown below and in Figure 2.1 over:

- Catenary zone, where the riser hangs in a catenary section
- Buried zone, where the riser is within a trench
- Surface zone, where the riser rests on the seabed and is a static pipeline or flowline.
2.2.2 Loading on a SCR

The vessel from which the SCR hangs is generally a floating production vessel, and as such is subject to wave, current and wind loading as described by 2H Offshore (2002b). The SCR connects to the vessel via either a flex joint or a taper stress joint. These transfer the dynamic motions of the vessel directly to the top of the SCR, which causes the TDP to move along the riser. It has been found that of all the vessel motions, heave causes the greatest stress fluctuations at the TDP as noted by Chandwani & Larsen (1997) and Chaudhury (2001).

The main forms of vessel motions are described below:

- First order motions – wave frequency motions caused by wave action on the vessel.
- Second order motions – low frequency motions caused by swell waves and light winds, often referred to as slow drift motions.
- Static offset – displacement resulting from mean environmental loads such as currents, waves and winds, or system failures, such as failed mooring lines.

In addition to the vessel motions the current and wave particle motions act directly on the SCR. This causes the riser to flex in the direction of the current, and can
invoke high frequency vortex induced vibration (VIV) motions in the riser string, Blevins (1990).

2.2.3 Vessel Motions

During the life of a SCR the vessel or FPS supporting the SCR will be subject to many different motions from small wave height, low period day-to-day waves, large wave height long period storm waves to second order motions such as slow drift, failed mooring line offsets or well completion activities. If the vessel moves towards the flowline, termed near offset, as shown in Figure 2.2, the top angle (of the SCR to the vertical axis) reduces, which lowers the top tension, decreases the length of the catenary and lays more of the SCR on the seabed. This moves the TDP away from the flowline, towards the vessel. Conversely if the vessel moves away from the flowline, termed far offset, the top tension and top angle increase. This moves the TDP towards the flowline, away from the vessel, lifting more of the SCR from the seabed into the catenary zone. Motions that move the vessel out of the SCRs near-far plane are termed transverse motions.

![Figure 2.2 – Near, Nominal and Far Offsets](image-url)
2.2.4  Touchdown Zone

The TDZ is the section of the SCR that interacts with the seabed. The TDP continually moves along the riser; for a 0.356m diameter SCR small day-to-day surface waves can cause horizontal motions of around 2m (5.5 diameters), while during a storm wave the TDP motions can be 4-5m (11 to 14 diameters) vertically and 20m (55 diameters) horizontally as shown by 2H Offshore (1999a). These continuous TDP motions can cause the SCR to dig itself into a trench, which have been reported as being up to ten diameters in depth, 2H Offshore (2001c), Figure 2.3.

![Figure 2.3 – Sketch of Touchdown Zone](image)

A study conducted within the STRIDE JIP by 2H Offshore (1999a) and reported in a paper by Thethi & Moros (2001) shows the probability of the TDP location of a 14” outer diameter SCR in the Gulf of Mexico, Figure 2.4. A summary of the riser properties used in the analysis is given in Table 2.1. The study states that the TDP location is highly dependant on the second order motions. The study concluded that in the plane of the riser, or near-far axis, the TDP moves less than ±8m (±23 diameters) during 50% of the riser’s life with maximum TDP movements of −90m (257 diameters) and 70m (200 diameters) with a 100 year event. Transverse to the plane of the riser the TDP moves less than ±0.5m (1.4 diameters) from the nominal TDP during 99% of the riser’s life and the maximum transverse distance moved for a 100 year event is ±6m (17 diameters).
Table 2.1 – Summary of the Properties of the STRIDE II Gulf of Mexico SCR, 2H Offshore (1999a) and Thethi & Moros (2001)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Gulf of Mexico SCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outer Diameter, $D_O$</td>
<td>0.356m (14&quot;)</td>
</tr>
<tr>
<td>Wall Thickness, $t$</td>
<td>0.0175m (0.8&quot;)</td>
</tr>
<tr>
<td>Height of the Attachment Point, $z_A$</td>
<td>1190m (3904ft)</td>
</tr>
<tr>
<td>Top Angle to Horizontal, $\theta$</td>
<td>12.0$^\circ$</td>
</tr>
<tr>
<td>Internal Fluid Density, $\rho_F$</td>
<td>900 kg/m$^3$ (861b/ft$^3$)</td>
</tr>
<tr>
<td>In Service Weight in Water, $m_s$</td>
<td>117 kg/m (10.5lb/ft)</td>
</tr>
<tr>
<td>Bending Stiffness, $E_I$</td>
<td>$5.51 \times 10^7$ Nm$^2$</td>
</tr>
<tr>
<td>Seabed Assumption</td>
<td>Rigid surface</td>
</tr>
</tbody>
</table>

2.3 SCR Design

SCRs are designed to resist the effects of the environmental, geotechnical and fluid loads. Analysis is conducted to calculate the static and extreme stresses, the fatigue
life and the probability of the riser clashing with the vessel, mooring lines and 
eighbouring risers. The design of a SCR is an iterative procedure: results from an 
analysis step feed directly into all subsequent analysis steps (2H Offshore, 2002b).

Initial riser pipe sizing is conducted based on the static configuration. The initial riser 
configuration is assessed using extreme storm loads (modelled by regular sinusoidal 
waves representing 1 year, 10 year and 100 year return conditions), and accidental 
design load cases, such as a failed mooring line. The analysis results are then 
assessed, and if required the riser is re-sized, and subsequently re-analysed, to 
account for any extreme stresses.

The minimum fatigue life, which is discussed by Campbell (1999), is determined by 
combining first and second order fatigue damage with the fatigue damage from VIV. 
First and second order fatigue can be determined by analysing the SCR under 
random sea loads, represented by a series of irregular waves (statistical 
representations of individual wave conditions which exist during a given period, e.g. 
three hours), Barltrop & Adams (1991). Fatigue assessment methods such as 
rainflow counting using the method presented by the ASTM (1985) can be applied 
and the maximum fatigue damage, which corresponds to the minimum fatigue life, 
calculated from the stresses induced from first and second order loading. VIV fatigue 
damage is determined from the riser response to current loading. A factor of safety 
(typically 10) is applied to the calculated fatigue lives and this is compared to the 
required life for the production system. If the riser is assessed to fail (the calculated 
fatigue life being too low) fatigue reduction measures, such as strakes or fairings, can 
be employed to improve riser response. These, however, can change the dynamic 
response of the riser, and may increase the stresses in the system.

2.4 SCR Modelling

2.4.1 Overview of SCR Modelling

SCRs are designed using numerical models that determine the riser response to static 
and dynamic loading. SCRs are generally designed using finite element analysis 
(FEA) due to their complexity and dynamic nature. However they can also be
modelled using the catenary equation to determine the static riser configuration and to check the FEA results.

2.4.2 Finite Element Analysis

Finite element analysis is used to assess the static and dynamic SCR response. SCRs are typically analysed using a non-linear time domain finite element (FE) code, such as ANSYS (ANSYS, 2000) or FLEXCOM-3D (MCS, 2004). Linear and frequency domain FE codes can also be used to analyse SCRs but assumptions must be made, such as the TDP being modelled by a fixed pin boundary condition, which results in the TDP not being properly modelled.

SCRs can be modelled using a string of beam elements. Enough elements should be used so that the catenary and buried zones are modelled completely with a few elements in the surface zone, 2H Offshore (2002b). Since the sections of interest are the riser/vessel interface and the TDZ, the element mesh should be suitably refined at these sections; as a guide it is recommended that the elements at the TDP are at most 1m long (Bensimon, 1998).

The SCR boundary conditions are the connection to the vessel and the interaction with the seabed. The vessel can be modelled using response amplitude operators (RAOs) that represent vessel motions. The seabed is generally analysed as either a rigid or flexible surface with lateral and axial friction coefficients. More complex pipe/seabed interaction models can be created using combinations of non-linear springs, gap, control and damping elements.

Riser analysis can be conducted using many FE codes. For this research the two FE codes used were ANSYS and FLEXCOM-3D, which are described briefly below.

2.4.3 FLEXCOM-3D

FLEXCOM-3D, produced by MCS International, is a specialist state-of-the-art riser finite element code. The suite of programs can model all riser types using both linear (frequency domain) and non-linear (time domain) methods. Further details of these
FE methods can be found in Barltrop & Adams (1991). FLEXCOM-3D has modelling functions tailored to SCR analysis, allowing the model to be specified as a cable between the vessel and the seabed. This removes the need for initial complex analysis to create the SCR shape.

The program is limited in terms of pipe/soil interaction as the seabed is specified as a rigid or flexible surface. More complex forms of pipe/soil interaction can be achieved by using non-linear springs in combination with the rigid or flexible surfaces.

An advantage of using FLEXCOM-3D is the short runtime for static and dynamic analysis. A single extreme storm analysis with a 120s simulation time, for example, can be analysed within a five minute analysis runtime. This allows for comprehensive series of analyses to be conducted that reduces the need for severe load combination assumptions and could reduce the conservatism of a SCR design.

2.4.4 ANSYS

ANSYS, produced by ANSYS Inc, is a general purpose finite element code developed for many applications, including structural analysis, computation fluid dynamics and electromagnetic analysis. ANSYS has a specific pipe element that allows submerged structures to be modelled. The program is not tailored for riser analysis, instead it has the ANSYS Parametric Design Language (APDL) which allows RAOs and other complex functions to be created. SCRs can be modelled in ANSYS but there is no cable option so the riser must start as a straight pipe and be allowed to deform into the catenary shape. This is described in more detail in Section 9.0.

The rigid and flexible surfaces used in FLEXCOM to model the seabed may be created in ANSYS using a combination of either shell and contact elements or non-linear springs. More complex pipe/soil interaction models can be developed using non-linear springs, gaps, damping and control elements that are native to ANSYS.
The runtime for an extreme storm analysis in ANSYS is around 1.5 hours. For this reason ANSYS is not used as the day-to-day riser analysis package, being reserved for more complex interaction problems.

2.4.5 Catenary Equation

The catenary equation was developed to find the shape of a suspended cable with uniform mass and was first proposed by Jacques Bernoulli in 1691. It assumes that the cable/riser has no external loading, such as current loads, and has no bending stiffness. This assumption is applicable to SCRs as they have a high aspect ratio (length divided by diameter) and, as shown by Pesce et al (1997) the global dynamics of SCRs are dominated by their geometric rigidity. The catenary equation is only applicable in the catenary zone of the SCR as it does not account for the interaction with the seabed. The fundamental equation of a catenary is derived from the following equation from Bernoulli (1691):

$$ a \frac{d^2 z}{dx^2} = \left[1 + \left(\frac{dz}{dx}\right)^2\right]^{1/2} $$

(2.1)

where

- \(a\) is a constant
- \(x\) is the horizontal distance
- \(z\) is the vertical distance

This can be solved into the standard ‘catenary’ equation, shown in equation (2.2) which allows the calculation of the height above the seabed of a point on the riser given the distance from the TDP, the horizontal tension at the TDP and the submerged mass per unit length. A graphical representation is given in Figure 2.5. The symbols used in the equations are explained overleaf in Figure 2.5.

$$ z = \frac{H}{m_s g} \left[ \cosh\left(\frac{m_s gx}{H}\right) - 1 \right] $$

(2.2)

In addition a second equation has been derived to describe the relationship between the tension at a point on the riser and the submerged mass per unit length, the height above the seabed of that point and the horizontal tension at the TDP, equation (2.3). The derivation of this equation is given in detail in Appendix A.
From equations (2.2) and (2.3) it is possible to create a set of equations that describe the extremities of the SCR if the height of the attachment point and the top angle are both known. Details of the derivation of these equations conducted by the writer are given in Appendix A and summarised below:

\[
\alpha_{TOP} = 90 - \theta 
\]  \hspace{1cm} (2.4)

\[
x_{TDP} = z_A \frac{\text{arcsinh} [\tan \alpha_{TOP}]}{\cosh (\text{arcsinh} [\tan \alpha_{TOP}]) - 1} = z_A \frac{\text{arcsinh} [\tan \alpha_{TOP}]}{\sqrt{\tan^2 (\alpha_{TOP}) + 1} - 1} 
\]  \hspace{1cm} (2.5)

\[
S_R = z_A \frac{\tan \alpha_{TOP}}{\cosh (\text{arcsinh} [\tan \alpha_{TOP}]) - 1} = z_A \frac{\tan \alpha_{TOP}}{\sqrt{\tan^2 (\alpha_{TOP}) + 1} - 1} 
\]  \hspace{1cm} (2.6)

**Figure 2.5 – Notation for SCR**

where

- \( g \) acceleration due to gravity
- \( H \) horizontal force in the riser, and the tension at the TDP
- \( m_s \) submerged mass per unit length
- \( M_{TDP} \) bending moment at the TDP
- \( S \) length of riser from the TDP to a point on the riser
S\textsubscript{R} length of riser between the TDP and the vessel
T tension at a point along the riser
T\textsubscript{TOP} top tension
x horizontal distance between the TDP and a point on the riser
x\textsubscript{TDP} horizontal distance between the vessel and the TDP
z vertical distance from the seabed to a point on the riser
z\textsubscript{A} vertical distance between the riser attachment point and the seabed
\( \alpha \) angle between the catenary and the horizontal axis
\( \alpha\textsubscript{TOP} \) angle between the riser/vessel connection and the horizontal axis
\( \theta \) top angle. The angle between the vessel/riser connection and the vertical axis

Once the basic dimensions of the SCR are known the tension and bending moment along the static riser can be determined. Tension along the riser can be calculated using equation (2.3). The bending moment distribution in the catenary zone is derived from the non-linear, large deflection curvature relationship given below:

\[
k = \frac{\frac{d^2 z}{dx^2}}{\left[1 + \left(\frac{dz}{dx}\right)^2\right]^\frac{3}{2}} \tag{2.7}
\]

where

\( k \) curvature

Substituting equation (2.2) into equation (2.7), and differentiating gives:

\[
k = \frac{\frac{m_s g}{H} \cosh\left(\frac{m_s g x}{H}\right)}{\left[1 + \left[\sinh\left(\frac{m_s g x}{H}\right)\right]^2\right]^\frac{3}{2}} \tag{2.8}
\]

This can be used with the relationship between bending moment and curvature given in equation (2.9), to calculate the bending moment at any point on the riser in the catenary zone.

\[
M = -kEI \tag{2.9}
\]
where

\begin{align*}
E & \quad \text{Young's modulus} \\
I & \quad \text{second moment of area}
\end{align*}

Generally the top of the riser has the highest tension and lowest bending moment while the TDP has the lowest tension and the highest bending moment. Examples of slope, tension, shear force, bending moment and von Mises stress distribution along a SCR in 1800m water depth with a 12° top angle and 12.75” (0.324m) outer diameter calculated using the catenary equations and FEA are given in Figure 2.6 to Figure 2.10 respectively. Further details of this riser are given in Appendix A. Equations for the tension at the top of the riser and the tension, bending moment and curvature at the TDP are given below:

\begin{align*}
\text{Tension at the vessel/riser interface} & \quad H = \frac{S_R m_s g}{\tan \alpha_{\text{TOP}}} \quad (2.10) \\
\text{Tension in the SCR at the TDP} & \quad T_{\text{TOP}} = \frac{H}{\cos \alpha_{\text{TOP}}} \quad (2.11) \\
\text{Curvature at the TDP} & \quad k_{\text{TDP}} = \frac{m_s g}{H} \quad (2.12) \\
\text{Bending moment at the TDP} & \quad M_{\text{TDP}} = \frac{-m_s g}{H}EI = \frac{\tan \alpha_{\text{TOP}}}{S_R}EI \quad (2.13)
\end{align*}

Note that the catenary equation calculates the shear force at the TDP to be 0.0kN.
Figure 2.6 – Example SCR Slope Calculated using FE and Catenary Equations

Figure 2.7 – Example Effective Tension Along SCR
Shear force from Catenary equations is 0kN, while FE predicts 10kN.
The catenary and FE solutions correlate well; however there are differences in the calculated shear force, bending moment and stress near the TDP. The differences between the FE and catenary solutions have been examined by Pesce et al (1997). They showed that in a SCR the bending stiffness is significant at the ends of the riser, specifically at the TDP. Pesce et al (1997) defined the flexural length parameter, $\lambda_L$, which they determined to be the distance between the actual TDP and the FEA solution TDP. The flexural length parameter is based on the work by Love (1927), where he shows that the bending stiffness of a taught wire suspended between two points, known as the “beam-string” problem, is only significant at the ends of the wire. The flexural length parameter defines the excess length in the wire, calculated by ignoring the bending stiffness. A subscript of L has been added to the flexural length parameter in this thesis to distinguish it from other parameters.

$$\lambda_L = \sqrt{\frac{EI}{H}}$$ (2.14)

The effect of current drag on catenary structures has been investigated by many authors including Zajac (1957), Pedersen (1975) and Vas & Patel (2000) who developed the catenary equations for laying subsea telecommunications cables. Solutions to the global static problem of the catenary shape that includes the bending
stiffness and current loading was conducted by Pesce et al (1998); however this formula only works for low current velocities.

The catenary equation provides a simple means to understand the load distribution in a SCR. More sophisticated methods are needed to evaluate the forces on the SCR in the TDZ.

2.4.6 TDZ Models

SCR analysis is conducted using rigid or flexible surfaces to represent the seabed. Palmer (2000) describes the differences between the SCR/seabed forces for horizontal rigid, elastic and rigid/elastic seabeds. These seabed models are described below, with the initial definition sketch given in Figure 2.11.

In the catenary zone the curvature of the riser is determined by the interaction between the tension and the submerged weight as shown by equation (2.8) and the riser can be considered to be a string for modelling purposes. In the buried and surface zones the riser is a beam resting on a surface. In between these two zones is the TDZ that encompasses the interface between the riser-string in the catenary zone and the riser-beam in the buried zone.

![Figure 2.11 – Touchdown Zone Definition Sketch](image)

An overview of the forces on a horizontal rigid seabed is given in Figure 2.12. At the TDP the bending moment reduces from the catenary bending moment to zero and there is a concentrated reaction force, which from Pesce et al (1998) and Palmer (2000), is given as:
Rc = \sum \sqrt{ \frac{EI}{H} } \tag{2.15}

where

Rc \quad \text{is the concentrated reaction at TDP assuming a rigid surface}

The reaction of the pipe lying on the seabed is given as:

Rp = m_r g \tag{2.16}

where

Rp \quad \text{is the reaction along the pipe}

For the example riser shown previously and described in Appendix A the TDP reaction force is 10.0kN. Using equation (2.16) the TDP reaction force is calculated to be 10.4kN, a difference of 0.4kN. This shows that for a rigid seabed the FE models are in close agreement with closed form solutions.

An overview of the interaction forces between a SCR and the seabed assuming the seabed is elastic and horizontal is given in Figure 2.13. In the catenary zone the curvature is described by equation (2.8). The curvature in the surface zone is zero, and the reaction force, Rp, described by equation (2.16). Either side of the TDP is a transition zone where the TDP reaction decreases to zero in the catenary zone and Rp in the surface zone. Palmer (2000) stated that the length of the transition zones depends on the flexural rigidity of the riser and the relationship between the reaction force and vertical displacement into the seabed. This relationship is similar to the flexural length parameter, \lambda_L, given by Pease et al (1997).
The riser in the buried and surface zones on an elastic seabed can be considered to be a beam on an elastic foundation and solutions can be found using the work by Hetenyi (1946) who produced a series of closed form solutions that can be adapted to model the riser/seabed interaction. This analysis is covered in later sections of this thesis.

![Figure 2.13 – SCR with Elastic Seabed](image)

If the seabed is assumed to be a plastic horizontal surface the interaction forces between the seabed and the riser will be similar to the elastic seabed. However, any deformation into the seabed will be unrecoverable. This has a greater effect when the riser is moved into a new position, as the deformations in the seabed will alter the stress distribution along the buried zone of the riser.

Closed form methods used to analyse SCRs are developed around the catenary equations. For small vessel motions Pesce *et al* (1997) showed that the TDP could be treated as an articulation. However they also showed that for larger vessel motions the riser bending stiffness is required in TDZ models. They also defined a non-dimensional soil rigidity parameter, \( K_R \), shown below, which enabled them to develop a series of non-dimensional diagrams relating the elastica (elastic line), horizontal angle, shear effort and curvature as functions of a non-dimensional arc length parameter, \( S/\lambda_L \). A subscript of R has been added to the non-dimensional soil rigidity parameter in this thesis to distinguish it from other parameters. This solution is given below.

\[
K_R = \frac{kEI}{H^2} = \frac{k\lambda_L^2}{H} = \frac{k\lambda_L^4}{EI}
\]  

(2.17)
The effect on the distribution of stress, and hence fatigue life, around the TDP was assessed by Langner (2003). He hypothesised that if the trench was catenary shaped the reaction force at the TDP would be spread and the fatigue damage lower compared to a rigid seabed equivalent. This model indicates the benefits of using the trench in SCR analysis, however real SCR trenches are not catenary shaped, instead being teardrop, or ladle shaped as observed by the writer in Bridge et al (2003).

The effect on the reaction forces at the TDP when the SCR moves between the near and far offsets is shown in Figure 2.14 where the seabed is assumed to be plastic. Any deformation, and hence reaction forces developed will remain when the riser moves from one offset to another. Eventually the reaction force in the TDZ will be equal to $R_c$, and a trench in the seabed will have been formed. It is important to note that this is one of many possible SCR trenching mechanisms, which will be examined in this thesis.

![Figure 2.14 – SCR with Plastic Seabed](image)

2.4.7 Summary

Catenary equations provide a quick method to determine the geometric properties and static loads on a SCR. The specialist finite element analysis program is used for the bulk of the SCR design as many analyses can be run and post-processed quickly, but simplifications must be made at the TDP. The general purpose FEA code is used for confirmation of the simplifications made in the specialist riser program, and for more complex pipe/soil interaction models.
2.5 Subsea Soils

Typical sediments that are found in deep-water environments are likely to be very soft or silty clays as shown by Yen et al (1975), Dunlap et al (1990) and Sage Engineering Ltd (1998). Since SCRs are typically used in water depths that exceed 300m only saturated clay soils ($S_r = 1$) are considered within this thesis.

Ho (1988) reported that deep ocean sediments have an overall behaviour that is similar to terrestrial soils. This conclusion was based upon a comparison of undrained shear strength, compression, consolidation, and stress-strain behaviour. Studies conducted on deep water sediments from the North Atlantic abyssal plains by Boudet & Ho (2004) show that the sediments have high sensitivities, but are similar to terrestrial soils. These papers indicate that any testing programs conducted using on-shore saturated clays are applicable to deepwater seabeds and SCRs. One property of soil profiles taken from projects in the Gulf of Mexico is that the topmost soil layer is observed to be stiff crust approximately 1m to 3m thick, 2H Offshore (2001c).

The main properties used to describe a subsea clay soil are the undrained soil shear strength, $S_u$, plasticity index, $I_p$, submerged unit weight of the soil, $\gamma_s$, Poisson's ratio, $\nu$ (typically equal to 0.5 for undrained conditions) and the voids ratio, $e_s$. Undrained shear strength is the resistance to failure of the soil by shear stress over a 'short' time, which for clays is typically measured in hours and days. Typically near surface subsea soils in deepwater areas have an undrained shear strength of $< 20kPa$ and may be defined as 'very soft' in accordance with BS 8004 (1986). The undrained shear strength is also observed to increase with depth below the seafloor and can be written as a function in the form given below.

$$S_u = S_{uo} + S_{ug}z$$

where

$S_{uo}$ is the undrained shear strength at soil surface

$S_{ug}$ is the undrained shear strength gradient

$z$ is the depth below the surface
Clay soils are sensitive to remoulding, suffering considerable loss of strength due to their natural structure being damaged or destroyed. The sensitivity of clay is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength. Typical values for the sensitivity of soft subsea clays are reported by Bostrom et al (1998) to be between 2 and 4.

The plasticity index of a clay soil is the difference between the liquid limit and the plastic limit. It describes the amount of water the soil can hold before becoming a liquid. It was found by Olsen et al (1982) that the plasticity index in sediment typically found in deepwater sediments is high, being greater than 30%. The maximum plasticity index measured by Olsen was around 80%. However plasticity index’s around 90% have been reported by Quiros & Little (2002) in deepwater marine sediments. The plasticity index has been correlated to the undrained shear strength of clay soils, for which Skempton (1951) proposed a relationship between the ratio of undrained shear strength and effective vertical stress and plasticity index. This is given below.

\[ \frac{S_u}{\sigma_v} = C_1 I_p + C_2 \]  (2.19)

where

- \( \sigma_v \) is the effective vertical stress
- \( I_p \) is the plasticity index
- \( C_1 \) and \( C_2 \) are factors that have been found experimentally and are given in Table 2.2.

### Table 2.2 – Factors C1 and C2 Relating Plasticity Index to Undrained Shear Strength

<table>
<thead>
<tr>
<th>Source</th>
<th>C_1</th>
<th>C_2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Skempton (1951)</td>
<td>0.0037</td>
<td>0.11</td>
</tr>
<tr>
<td>Ladd (1981)</td>
<td>0.0008</td>
<td>0.21</td>
</tr>
<tr>
<td>Quiros (2002)</td>
<td>0.0008</td>
<td>0.199</td>
</tr>
</tbody>
</table>
Typical undrained shear strength and plasticity index for deepwater in the Gulf of Mexico, West Europe and West Africa are given in Table 2.3. Examples of the plasticity index, submerged unit weight of soil, Poisson’s ratio and voids ratio for different clay types (very soft through to hard) taken from DNV (1998) are given in Table 2.4.

### Table 2.3 – Typical Geotechnical Parameters by Region, Fugro (1999)

<table>
<thead>
<tr>
<th>Deepwater Area</th>
<th>$S_{uo}$ (kPa)</th>
<th>$S_{ug}$ (kPa/m)</th>
<th>$I_p$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gulf of Mexico (lower Bound)</td>
<td>1.2</td>
<td>0.8</td>
<td>50</td>
</tr>
<tr>
<td>Gulf of Mexico (typical)</td>
<td>2.6</td>
<td>1.3</td>
<td>50</td>
</tr>
<tr>
<td>West Europe (West of Shetland / Norway)</td>
<td>2.4</td>
<td>1.6</td>
<td>-</td>
</tr>
<tr>
<td>West Africa</td>
<td>1.9</td>
<td>1.2</td>
<td>100</td>
</tr>
</tbody>
</table>

### Table 2.4 – Typical Geotechnical Parameters for Clay from DNV (1998)

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Shear Strength</th>
<th>Bulk Density $\gamma_s$ (kN/m$^3$)</th>
<th>Poisson’s Ratio $\nu$</th>
<th>Void Ratio $e_s$</th>
<th>Plasticity Index $I_p$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>5</td>
<td>4.4</td>
<td>0.45</td>
<td>2.0</td>
<td>60</td>
</tr>
<tr>
<td>Soft</td>
<td>17</td>
<td>5.4</td>
<td>0.45</td>
<td>1.8</td>
<td>55</td>
</tr>
<tr>
<td>Stiff</td>
<td>70</td>
<td>7.4</td>
<td>0.45</td>
<td>1.3</td>
<td>35</td>
</tr>
<tr>
<td>Hard</td>
<td>280</td>
<td>9.4</td>
<td>0.45</td>
<td>0.8</td>
<td>20</td>
</tr>
</tbody>
</table>

2.6 Pipe/Soil Interaction

Research has been conducted into the area of soil-structure interaction. The majority of the references found concern either static problems, such as foundations, or high frequency problems, such as earthquakes. Due to the nature of the wave and current
loading on the floating production system and the riser, the frequency of SCR soil-structure interaction is around 0.1Hz. This frequency is lower than in high frequency problems, but too high to be considered a static problem.

2.6.1 Force/Displacement Model

The current practise for SCR finite element analysis is to use pipe/soil interaction models that were developed for buried pipelines and strip foundations. Simulations of risers and pipelines use simplified pipe/soil interaction models that are described in terms of non-linear elastic load/displacement curves as shown in Figure 2.15 from Chen & Han (1985). The model has zero force at zero displacement, as the displacement increases the load also increases linearly until a maximum load is reached where the load becomes constant with changing displacement. The maximum load is determined by the backbone curve, Idriss et al (1978), which represents the soil resistance to pipe embedment into a virgin soil. Typically backbone curves are constructed using bearing capacity theory. Simplifications are made in the numerical models to take into account the axial, horizontal and vertical downwards motions. Most models assume that the vertical upwards motions produce no force so objects are free to lift off of the seabed. Each degree of freedom, X, Y, and Z, has its own load/displacement curve that are described below and illustrated in Figure 2.15.

- Vertical (Z-axis) are based on bearing capacity and denoted as q-z curves
- Horizontal (Y-axis) are a combination of friction and passive soil resistance and denoted as p-y curves
- Axial (X-axis) are friction only and denoted as t-x curves
2.6.2 Soil Stiffness

Many FEA programs require a linear representation of the non-linear pipe/soil interaction curves to model the seabed. However, it was concluded by Fourie & Beer (1989) that linear soil stiffness models that neglect the possibility of soil yielding may produce inaccurate estimates of pipeline stress. Linear soil stiffness can be defined as the ultimate bearing load divided by a distance, as shown below:

\[ K = \frac{F}{\Delta} \]  

(2.20)

where

- \( K \) is soil stiffness per unit length
- \( F \) is force per unit length
- \( \Delta \) is displacement

There are a number of different methods that can be used to characterise soil stiffness, which include the Young’s modulus and the secant and tangent stiffnesses, Barltrop & Adams (1991). These are outlined below and illustrated in Figure 2.16:

- Young’s Modulus – the slope of the tangent force/displacement curve at the origin
- Secant stiffness – the average stiffness between two points, generally the origin and the point in question
- Tangent stiffness – the stiffness tangential to the point in question. Typically used to model soil stiffness when the displacements are small.

![Soil Resistance, F](image)

**Figure 2.16 – Non-linear Soil Model Showing Young’s Modulus, Secant and Tangential Stiffness**

### 2.6.3 Young’s Modulus of Soil

An estimation of the modulus of elasticity of a soil can be found using the formula below (D’Appolonia et al, 1971):

\[
E = \beta \times S_u
\]  \hspace{1cm} (2.21)

where

\[\beta\] is a dimensionless parameter determined experimentally

The dimensionless parameter \(\beta\) has been found to increase with increasing over consolidation ratio (OCR) while it decreases with high plasticity index and high organic content of the soil. D’Appolonia et al (1971) reported a range of soil elastic modulus and the associated \(\beta\) parameters from case studies of immediate settlement from plate bearing tests in undrained inorganic saturated clays with moderate to high plasticity. This shows that \(\beta\) ranges between 400 and 2500 with a mean of
approximately 1300. Smits (1980) suggested that for rough preliminary calculations of Young’s modulus for cyclic displacements $\beta$ could be taken as 400 for clay.

Further research conducted by Duncan & Buchignani (1976) correlated $\beta$ with the plasticity index and OCR as shown in Figure 2.17. They showed that $\beta$ decreases with increasing plasticity index and OCR. This finding is also reported by Taiebat & Randolph (2001).

![Figure 2.17 - Modulus of Elasticity of an Undrained Clay with Plasticity Index and Normalised Over Consolidation Ratio, Duncan & Buchignani (1976)](image)

For sand, Smits (1980) suggested the relationship given below.

$$E = 500\sqrt{\sigma^* v} p_a$$  \hspace{1cm} (2.22)

where

- $\sigma^* v$ is effective overburden pressure
- $p_a$ is atmospheric pressure
The shear modulus can also be used to represent soil stiffness. For small displacement dynamic analysis purposes the shear modulus, $G$, is independent of the drainage conditions and the loading cyclic period.

$$G = \frac{E}{2(1 + \nu)} \quad (2.23)$$

The shear modulus can be described in terms of the undrained shear strength by substituting the Young's modulus with equation (2.23) and assuming a value for Poisson's ratio ($\nu = 0.45$) from Table 2.4. This gives:

$$G = 0.345\beta \times S_u \quad (2.24)$$

It has been reported on many occasions that soils do not behave in a linear fashion. The resistance force in the soil mobilised due to pipe/soil interaction is also dependant on many factors, including loading history (hysteresis), consolidation time, consolidation load and interaction rate.

### 2.6.4 Rate of Failure of Soil

It has been demonstrated that the undrained shear strength increases as the rate of failure increases. Taylor (1943) showed that if the strain rate was increased from 0.001% per minute to 1% per minute the undrained shear strength of a clay increased by about 25%. Casagrande & Shannon (1949) examined a variety of cohesive materials, including Cambridge Clay (100% increase in $S_u$ when the time to failure was reduced from 465 seconds to 0.02 seconds) and Cucaracha Shale (60% increase in $S_u$ when the time to failure is reduced from 1000 seconds to 0.05 seconds) as shown in Figure 2.18. On this basis Casagrande & Shannon (1949) concluded that the stiffness of a soil could increase by 100% under dynamic loading conditions.
2.6.5 Consolidation

Consolidation refers to the time dependant dissipation of pore water in a soil and the subsequent volume change of the soil. Consolidation generally occurs when a load, such as a pipeline, is placed on top of the soil. The effects of consolidation have been examined by many authors, including Brennoden & Stokkeland (1992) who examined a pipe lying on a marine sediment. They showed that as the consolidation time and consolidation weight are increased the settlement depth and soil shear strength both increase as the soil volume reduces. This indicates that if a SCR is left in one position for a length of time the soil strength beneath the riser, and hence soil stiffness will increase.

2.6.6 Cyclic Loading

Environmental loading on the vessel and SCR causes the TDP to move and creates dynamic, or cyclic, loading on the seabed. Many investigations have been conducted into cyclic loading of gravity foundation of offshore structures (Anderson, 1991) and buried pipelines (Audibert et al, 1984) but little work has been conducted on SCR cyclic pipe/soil interaction.
The paper by Andersen (1991) describes the foundation design of offshore gravity structures. He suggests that a modified bearing capacity method can be used to model combined static and cyclic structure/foundation interaction and that the combined static and cyclic soil strength is significantly lower than the static soil strength. However, Andersen's model is designed to help calculate failure mechanisms over long periods of time where the structure sinks into the seabed due to consolidation and cyclic loading. This is a possible SCR trenching mechanism where the riser TDP will continue to penetrate into the soil due to pipe self weight and cyclic motions generated from vessel/wave motions and current/riser (VIV) loading.

In-contact cycling of buried pipelines, where the pipeline does not lose contact with the soil, has been written about by many authors including Barltrop & Adams (1991), Dunlap et al (1990) and Nova (1981). They show, similarly to Andersen (1991) above, that cyclic motions soften the seabed. The effect is that as the number of cycles increase the soil stiffness reduces. Further work by Dormieux & Pecker (1995) on cyclic soil stiffness during earthquakes has shown that soil inertia forces may be neglected in high frequency models, which could be applicable to high mode VIV loading.

2.7 Vertical Downwards Pipe/Soil Interaction

2.7.1 Riser Penetration into the Seabed

The pipeline penetration depth reported by Bostrom et al (1998) is primarily a function of the riser weight, riser diameter, soil shear strength and load history. Riser penetration is generally modelled by the backbone curve, as described below.

The backbone curve gives the maximum soil resistance force to pipe penetration at a given depth in a virgin soil. A pipe resting on a seabed can be considered as a strip foundation. The soil loading pressure from a strip foundation can be calculated using bearing capacity theory. The bearing capacity of a strip foundation in an undrained clay soil was defined by Terzaghi (1943) and is given below:

\[ q_U = N_c S_U + \gamma z \]  

(2.25)
where
\[ q_u \] is the ultimate bearing capacity (pressure)
\[ N_c \] is the shape and depth factor
\[ S_u \] is the undrained soil shear strength
\[ \gamma \] is the unit weight of the clay soil, for submerged soils this is taken as the submerged unit weight of the soil, \( \gamma_s \)
\[ z \] is the depth of the foundation below seabed (to bottom of pipe)

The \( \gamma z \) term within equation (2.25) is applicable only in foundations that are not backfilled with material, i.e. in the case of a SCR when the riser is sitting in an open trench. Skempton (1951) defined values of \( N_c \) with depth based on both experimental and theoretical results. The equation defining these values is given below.

\[
N_c = \min \left[ 5.14 \left( 1 + 0.23 \sqrt{\frac{z}{D}} \right), 7.5 \right]
\]

Meyerhof (1963) produced an equation for the bearing capacity shape and depth factor based on a plasticity solution of a smooth strip footing at the bottom of an unsupported trench. This equation is given below.

\[
N_c = 5.14 \left( 1 + 0.23 \frac{z}{D} \right)
\]

Murff et al (1989) presented exact upper and lower bound equations for smooth and rough surfaced pipes penetrated to half a diameter depth in clay soil. These solutions are complex but can be simplified down to a non-dimensional equation relating the bearing force to the outer diameter of the pipe and the shear strength, as shown below. For a pipe embedded to half a diameter depth the upper bound solutions are similar to those given by Skempton (1951) for the rough pipe and Meyerhof (1963) for the smooth pipe. A summary of this comparison is given in Table 2.5.

\[
N = \frac{Q}{S_u B}
\]
Table 2.5 – Factors of N for Pipe at Half Diameter Embedment

<table>
<thead>
<tr>
<th>Reference</th>
<th>Bearing Capacity Factor, N at Embedment ( \frac{D}{2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Smooth Pipe – 4.00</td>
</tr>
<tr>
<td></td>
<td>Smooth Pipe – 5.56</td>
</tr>
<tr>
<td>Skempton (1951)</td>
<td>5.98</td>
</tr>
<tr>
<td>Meyerhof (1963)</td>
<td>5.65</td>
</tr>
</tbody>
</table>

### 2.7.2 Static Pipe Penetration Depth

The static penetration depth of a SCR into a clay soil can be determined using the ultimate bearing load, which is calculated from the bearing capacity formulae. This is discussed by Small et al (1971) and shown below.

\[
Q_U = q_u LB
\]  

(2.29)

where

- \( Q_U \) is the ultimate bearing load
- \( L \) is the length of foundation
- \( B \) is the bearing width, which is equal to the pipe diameter for pipe penetrations greater than half a diameter, otherwise \( B \) is calculated using the equation below. The derivation of this equation is given in Appendix B.

\[
B = 2\sqrt{Dz - z^2}
\]  

(2.30)

The method is discussed by Small et al (1971) and Bostrom et al (1998) and assumes that the pipe will sink until the bearing resistance equals the submerged pipe weight. The equation used is for the back filled trench that is given in equation (2.25).

\[
m_s g = Q_U = N_c S_u B
\]  

(2.31)
Since the soil shear strength can be represented as a function of depth as shown in equation (2.18), the above equation can be rearranged into an equation for the static embedment depth as shown below.

\[ z = \frac{1}{S'_{ug}} \left( \frac{m \cdot g}{N_c \cdot B} - S_{0u} \right) \]  

(2.32)

The relationship between pipeline embedment and the applied force was examined by Dunlap et al. (1990) using an experiment that pushed a 0.1524m diameter pipe into a remoulded clay sediment at different penetration speeds up to a penetration depth of 0.6 diameters. They derived an equation to estimate pipe embedment under monotonic loading, which is given below. This shows that penetration displacement is proportional to the penetration force and inversely proportional to a power of the penetration speed, the undrained soil shear strength and pipe diameter.

\[ z = \frac{0.01573}{D^{0.7822}} \left( \frac{F}{S_u \cdot \left( \frac{V}{D} \right)^n} \right)^{1.7822} \]

(2.33)

where

- \( z \) is the penetration depth (inch)
- \( F \) is the applied force (lbs)
- \( D \) is the pipe diameter (inch)
- \( V \) is the push in velocity (inch/s)
- \( S_u \) is the soil shear strength in (lbs/ft)
- \( n \) is the visco-elastic rate constant, taken as 0.03 for the sediment examined (dimensionless)

Similar experiments where a pipe was monotonically penetrated into a clay soil up to a depth of half a diameter were conducted by Verley & Lund (1995). They developed an equation for static penetration that weights the importance of pipe self weight and soil shear strength (parameter S) and soil shear strength and soil unit weight (parameter G). Using a spread sheet program they determined that the penetration data obtain during their experiments was best represented by the following empirical equation:

\[ \frac{Z}{D} = 0.0071 \left( SG^{0.3} \right)^{1.2} + 0.062 \left( SG^{0.3} \right)^{0.70} \]

(2.34)
where

\[ S = \frac{m_s g}{D S_U} \]  

(2.35)

\[ G = \frac{S_u}{D \gamma_s} \]  

(2.36)

The equations by Dunlap et al (1990) and Verley & Lund (1995) improve the available models for shallow pipe penetration (less than half a diameter) into clay soils, however they require additional validation before they can be used for pipe penetration above half a diameter.

After initial static penetration of the pipe, the pipe will continue to sink into the soil due to consolidation. The long term penetration of the pipe, assuming there is no dynamic loading, can be determined from the summation of the initial static penetration and the time dependant settlement theory, Craig (1996). The time dependant settlement is proportional to the square root of the time factor, \( T_v \), which is given below.

\[ T_v = \frac{c_v t}{d^2} \]  

(2.37)

where

- \( c_v \) is the coefficient of consolidation (m²/year)
- \( t \) is the time at which the pipe penetration is required (years)
- \( d \) is the drainage distance, equivalent to the pipe diameter (m)

Other aspects of seabed soil interaction that need to be considered in design are vertical and lateral cyclic loading. Lateral cyclic loading of a pipe in a soft clay soil was investigated by Morris et al (1988). They showed that pipes penetrate further into clay soils with increasing number of lateral pipe/soil cycles and cyclic loads. They produced a number of non-dimensional curves that relate the number of lateral
cycles, the normalised force and the penetration depth. These findings were confirmed by the work conducted by Bostrom et al (1998).

2.7.3 Static Stiffness

Static soil stiffness is used to model static pipe penetration. In riser analysis models a linear stiffness is typically used to represent the backbone curve. The simplest method is to use the bearing capacity equations and calculate the depth of pipe penetration for the given submerged pipe weight per unit length as given in DNV (1998) and shown by Chaudhury (2001). This is illustrated in Figure 2.19 with the resulting equation given below.

\[ K_s = \frac{m_s g}{z} = \frac{N_c S_u B}{z} \]  

(2.38)

where

\[ K_s \]

is the normalised static pipe/soil stiffness (dimensionless)

![Figure 2.19 – Static Soil Stiffness](image)

Static soil stiffness can also be calculated using elastic theory assuming linear stress-strain theory, as given by Craig (1992), and takes the following form.

\[ z = \frac{q B}{E (1 - v^2) f_s} \]  

(2.39)
where

\[ I_s \] influence factor that depends on the shape of the loaded area and is taken as 2 for a flexible strip footing on a quasi-elastic foundation.

This equation is combined with the ultimate bearing load equation (2.25) and rearranging in terms of stiffness gives:

\[
K_s = \frac{Q}{zL} = \frac{E}{2(1-\nu^2)} = \frac{G}{1-\nu}
\]

(2.40)

Using the soil data from Table 2.4 and equation (2.21), which relates Young’s Modulus to undrained shear strength the above equation simplifies into a linear relationship between static pipe/soil stiffness and undrained shear strength.

\[
K_s \approx \frac{5}{8} \beta S_u
\]

(2.41)

Both studies demonstrate that the static soil stiffness is a function of the undrained shear strength, and has similar values to the Young’s Modulus of the soil, which can be calculated using equation (2.21)

2.7.4 Stiffness After Initial Penetration

After the initial penetration of the pipe into the soil it has been observed in experimental studies by Bostrom et al (1998) that the soil will have plastically deformed around the pipe. If the penetration force is relaxed the force in the soil will reduce to zero and the soil will heave slightly, that is, expand and push the pipe upwards by a small distance. If the pipe is then pushed into the soil the force will increase from zero to the force defined by the backbone curve at that depth as shown in Figure 2.20. The distance over which the force changes from zero to the maximum is termed the mobilisation distance. In pipe/soil interaction the mobilisation distance is generally taken as a function of the outer diameter and in this thesis is represented by the symbol \( \Lambda \). The mobilisation distance is reported to be approximately 10% of the pipe diameter (\( \Lambda = 0.1 \)) by Audibert et al (1984) and Poulos (1988).
A more accurate representation of the pipe/soil interaction curve after the initial penetration is to use a hyperbolic force/displacement curve as suggested by Chaudhury (2001). This hyperbolic curve was based on the hyperbolic force/displacement interaction curve for sand developed by Audibert et al (1984) and is similar in form to the hyperbolic pipe/soil interaction curve developed by Hardin & Drnevich (1972) that was originally proposed for cohesive soils by Kondner (1963). The hyperbolic pipe/soil interaction curve scales the force/displacement curve using the ultimate bearing load from the backbone curve and the mobilisation distance. The hyperbolic model is generally given in the following form:

\[ Q = \frac{z}{A' + B'z} \]  

\[ A' = \frac{(1 - X)z_U}{Q_U} \]  

\[ B' = \frac{X}{Q_U} \]

where

\[ X \] is an empirical soil parameter that varies between 0.85 (for soft clays) to 0.93 (for stiff clays).
The force/displacement curve presented can be used to calculate soil stiffness for pipe/soil interaction after the initial penetration. These form the basis for dynamic pipe/soil interaction, as discussed below.

2.7.5 Dynamic Stiffness and Cyclic Loading

The effect of a pipe cycling in soft clay has been examined by many authors including Andersen et al (1978), Nova (1981) and Dunlap et al (1990). The effect is that as the number of cycles increases the stiffness reduces. This is illustrated in Figure 2.21, which is taken from Nova (1981) and shows that after 50 cycles the bearing stress reduces for the same strain level, indicating a reduction in soil stiffness.

![Figure 2.21 - Cyclic Loading, Nova (1981)](image)

The experiments to examine cyclic pipe/soil interaction of a surface pipeline by Dunlap et al (1990) used a 0.152m diameter pipe penetrated up to a depth of one diameter into a remoulded clay sediment. The pipe was then cycled for two hours around a mean static load so that the pipe was always in contact with the soil. The tests were conducted using both force and displacement controlled tests. Dunlap et al (1990) presented the results in the form of modified backbone curves for 1, 10, 100, 1000 and 3000 cycles using the equation below.
\[ K_C = A\left(\frac{z}{D}\right)^B \]  

(2.45)

where

\[ K_C \] is the normalised soil stiffness (dimensionless)

\[ A, B \] are empirically derived constants (dimensionless)

A summary of the empirical factors A and B determined by Dunlap et al (1990) are given in Table 2.6 and represented graphically as dynamic backbone curves in Figure 2.22 and Figure 2.23 for the load controlled and displacement controlled tests respectively. The tests show that as the number of cycles increased the pipe/soil interaction force decreases. This is consistent with the work by Dormieux & Peacker (1995) who stated that the backbone curve for cyclic loading has lower values of N due to inertia forces.

<table>
<thead>
<tr>
<th>Cycle</th>
<th>Load Controlled Tests</th>
<th>Displacement Control Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( K_C^+ )</td>
<td>( K_C^- )</td>
</tr>
<tr>
<td>1</td>
<td>8.332</td>
<td>0.615</td>
</tr>
<tr>
<td>10</td>
<td>8.350</td>
<td>0.612</td>
</tr>
<tr>
<td>100</td>
<td>8.324</td>
<td>0.631</td>
</tr>
<tr>
<td>1000</td>
<td>8.480</td>
<td>0.660</td>
</tr>
<tr>
<td>3000</td>
<td>9.330</td>
<td>0.710</td>
</tr>
</tbody>
</table>

\[ R \] – Correlation Coefficient
The degradation of the soil resistance force due to cycling was also examined by Idriss et al (1978). They related the reduction in the Young’s modulus of the soil to
the number of cycles and the amplitude of the developed shear strain. This relative reduction was expressed as a degradation factor, $D_E$, which is calculated as the ratio of the Young's modulus of soil after $n$ cycles to the Young's modulus of the first cycles. For uniform cyclic loading Idriss et al (1978) found that the degradation factor was proportional to the number of cycles to the power of a degradation parameter, as shown below.

$$D_E = n^{-t} \quad (2.46)$$

where

- $n$ is the number of cycles
- $t$ is the degradation parameter and varies between 0.01 and 0.6 as shown by Idriss et al (1978).

The equation for the degradation factor can be combined with the soil stiffness calculated using the hyperbolic pipe/soil interaction model to calculate a dynamic pipe/soil interaction stiffness.

An iterative method for determining the dynamic soil stiffness from the static soil stiffness based on energy conservation is discussed by Chaudhury (2001). He shows that the energy in the static force/displacement curve is calculated using the equation below:

$$E_s = K_{ST} z_u \frac{z_u}{2} \quad (2.47)$$

where

- $E_s$ is the energy in the force/displacement curve
- $K_{ST}$ is the tangent stiffness at the origin from Audibert et al (1984) hyperbolic force/displacement curve.

The dynamic stiffness is then calculated by assuming that the energy in the dynamic model is the same as that in the static model, which gives the following equation.

$$K_D = \frac{2E_s}{z_D^2} \quad (2.48)$$
Combining these two equations shows that this method for calculating the dynamic soil stiffness is the static soil stiffness multiplied by the ratio of the square of the static penetration depth to the square of the dynamic displacement, as shown below.

\[ K_D = K_{St} \frac{z_u^2}{z_d^2} \]  

(2.49)

This indicates that for small displacements the dynamic soil stiffness could be hundreds of times greater than the static soil stiffness. The dynamic soil stiffness approach given by DNV (1992) and DNV (1998) and for half space theory as discussed by Barltrop & Adams (1991) is also based upon static soil stiffness and is similar to the bearing load equation (2.40) but with an additional factor of 0.88 included as shown below.

\[ K_D = 0.88 \frac{G}{1 - \nu} \]  

(2.50)

The shear modulus, \( G \), is calculated using an empirical relationship originally developed by Hardin & Drnevich (1972) which incorporates the voids ratio, effective soil stress, over consolidation ratio (OCR) and plasticity index in the OCR exponent, \( k_s \), as shown below.

\[ G = \frac{1300(2.97 - e_s)^2}{1 + e_s} \sqrt{\sigma_s \text{OCR}^{k_s}} \]  

(2.51)

where

- \( e_s \) is the voids ratio
- \( \sigma_s \) is the effective soil stress calculated by
  \[ \sigma_s = 0.75\gamma_s B \]  

(2.52)
- \( k_s \) is an empirical coefficient and a function of the plasticity index

This equation is in conflict with the energy conservation method presented by Chaudhury (2001). The dynamic soil stiffness calculated is 88% of the static soil stiffness, which indicates that within published data and recommended practices there is a large variation in the values of dynamic soil stiffness that can be used. This knowledge gap is one of the key areas to be researched in this thesis.
2.8 Lateral Soil Resistance

2.8.1 Surface Pipelines

For a pipeline resting on the surface, or with shallow penetration depth into a clay soil any lateral motions are modelled using coulomb friction as shown in BS8010: part 3 (1993). The suggested lateral friction coefficients for non-cohesive soils, such as sand, and cohesive soils, such as clay, are given in Table 2.7. These values are similar to those determined by Lambrakos (1985) who conducted a series of in-situ lateral pipeline movement tests in shallow water (around 20m) in the Gulf of Mexico.

Table 2.7 – Typical Effective Coefficients of Lateral Friction for North Sea Applications, from BS8010: part 3 (1993)

<table>
<thead>
<tr>
<th></th>
<th>Min</th>
<th>Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-cohesive (e.g. sand)</td>
<td>0.5</td>
<td>0.9</td>
</tr>
<tr>
<td>Cohesive (e.g. clay, silt)</td>
<td>0.3</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Coulomb friction provides a convenient framework for lateral pipe/soil interaction. However the papers by Lyons (1973) and Karal (1977) both suggest that Coulomb friction is not valid for lateral pipe sliding on soft clays as the pipes will have penetrated into the seabed and therefore do not slide over the soil, but rather interact with the soil and mobilise friction and passive resistance forces. To account for this additional force Karal (1977) generated a series of modified friction coefficients that accounted for Coulomb friction, penetration depth, lateral distance moved and soil type. Compared to the general friction coefficients the modified friction coefficients were reported to be lower for pipes on clay soils.

A series of lateral pipe/soil interaction experiments that used a half pipe section mounted onto a rigid frame allowing either motion of force to be applied laterally and vertically were conducted by Wagner et al (1987). The pipe section was penetrated into the consolidated virgin clay soil that was contained within a large test tank. A formula was developed for modelling the soil resistance to lateral pipe motion. The equation consists of two resistance forces, one due to friction and the
second due to passive soil resistance, or soil cohesion, as shown below. This equation was also used to analyse the data from the similar experiments by Morris et al (1988) who noted that the equation was basically an empirical formula.

\[ p_U = p_F + p_R \]  \hspace{1cm} (2.53)

where

- \( p_U \) is the ultimate lateral soil resistance force
- \( p_F \) is the sliding resistance and can be represented using
  \[ p_F = \mu m_s g \]  \hspace{1cm} (2.54)
  where \( \mu \) is the coefficient of friction of the soil, assumed to be 0.2 for clays
- \( p_R \) is the lateral passive soil resistance and can be represented using
  \[ p_R = \beta S_v A \]  \hspace{1cm} (2.55)
  where \( \beta \) is an empirical passive soil resistance coefficient determined by Wagner et al and given in Table 2.8 (dimensionless)
- \( A \) is the characteristic area

Table 2.8 – Empirical Soil Passive Resistance Factors, Wranger et al (1987)

<table>
<thead>
<tr>
<th>Condition</th>
<th>Penetration Depth</th>
<th>( \beta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simple monotonic loading</td>
<td>x1</td>
<td>39.3</td>
</tr>
<tr>
<td>After many oscillations at lateral forces</td>
<td>x2</td>
<td>31.4</td>
</tr>
<tr>
<td>below the monotonic breakout load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>After many large dynamic oscillations</td>
<td>x3</td>
<td>15.7</td>
</tr>
</tbody>
</table>

The effect of soil resistance to the pipe moving laterally from thermal expansion was examined in the tests conducted by Brennodden & Stokkeland (1992). The tests used a half pipe section mounted onto a rigid frame allowing a constant force to be applied laterally and vertically, to represent the force from thermal expansion and the consolidation weight respectively. A friction coefficient of 0.2 was used and the existing equations modified to include empirical factors as shown below.
\[ p_U = k_{\text{max}} \left( \mu m_g + \beta_{\text{max}} S_U A / D \right) \]  \hspace{1cm} (2.56)

where

- \( k_{\text{max}} \) is an empirical coefficient depending on the break out velocity and pipe diameter. For the experiments by Brennodden & Stokkeland (1992) this value was 0.8.
- \( \beta_{\text{max}} \) is an empirical coefficient dependant on the undrained shear strength. For the experiments by Brennodden & Stokkeland (1992) this value was 1.47.

### 2.8.2 Buried Pipelines

Models for the lateral pipe/soil interaction force for buried pipelines in clay soils were discussed by Audibert et al (1984). They suggest the maximum lateral force can be modelled using a bearing capacity formulation where the horizontal bearing capacity factor \( N_{ch} \) accounts for soil friction increasing with penetration depth. This model was also presented by Chaudhury (2001) for use in SCR analysis. Audibert et al (1984) also suggests that a hyperbolic model, similar to the one used for vertical downward pipe/soil interaction can be used to model the p-y curve, scaled between the ultimate lateral force, \( p_U \), and the ultimate lateral displacement, \( y_U \). The equations for the horizontal force and the horizontal mobilisation distance are given below.

\[ p_U = S_U N_{ch} D \]  \hspace{1cm} (2.57)

where

- \( N_{ch} \) horizontal bearing capacity factor

\[ y_U = 0.03 \text{ to } 0.05 \times \left( z + \frac{D}{2} \right) \]  \hspace{1cm} (2.58)

### 2.8.3 Pile Foundations

A pipe resting on the seabed, or in a trench can be considered analogous to a pile foundation. That is any transverse pipe motion is resisted by the soil in the trench, and similarly the pile wall resists any lateral pile motion. A series of experiments were conducted by Matlock (1970) to examine lateral pile/soil interaction force/displacement curves of offshore piles for static and cyclic lateral loads. These
curves are scaled to fit any pile/soil interaction problem using a maximum lateral force and a unit displacement. Matlock's non-dimensional pile/soil interaction curves for static and cyclic loading given in Table 2.9, are compared with the hyperbolic model suggested by Audibert et al (1984) in Figure 2.24 and shows that there is consistency between the two models. The maximum lateral force was developed from bearing capacity theories for piles and uses the equations given below.

\[
P_u = \begin{cases} 
3S_u + \gamma z + J \frac{S_U z}{D} & 0 < z < z_R \\
9S_u & z > z_R 
\end{cases} 
\tag{2.59}
\]

where

- \(J\) is an empirical constant varying from 0.5 for Gulf of Mexico clays to 0.25 for stiffer clays.
- \(z_R\) is the depth below the soil surface to the bottom of the reduced stiffness zone and is calculated using the equations below:

\[
z_R = \frac{6D}{\gamma D / S_u + J} 
\tag{2.61}
\]

The unit displacement, \(y_c\), is calculated as a function of the pile diameter using the equations given below.

\[
y_c = 0.25 \varepsilon_c D 
\tag{2.62}
\]

where

- \(y_c\) is the unit displacement of the pile/soil interaction model
- \(\varepsilon_c\) is the strain occurring at 50\% of the maximum stress in a laboratory undrained compression tests. If laboratory data is not available then the values given in Table 2.10 by Barltrop & Adams (1991) should be used, Table 2.10.
Table 2.9 – Shape of Static and Cyclic Lateral Pile/soil Interaction Models for Soft Clays, Matlock (1970)

<table>
<thead>
<tr>
<th>Unit Displacement, y / y&lt;sub&gt;C&lt;/sub&gt;</th>
<th>Static Normalised Force, P / P&lt;sub&gt;U&lt;/sub&gt;</th>
<th>Cyclic Normalised Force, P / P&lt;sub&gt;U&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>1.0</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>3.0</td>
<td>0.72</td>
<td>0.72</td>
</tr>
<tr>
<td>8.0</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>15.0</td>
<td>1.0</td>
<td>0.72 z / z&lt;sub&gt;R&lt;/sub&gt;</td>
</tr>
<tr>
<td>20.0</td>
<td>1.0</td>
<td>0.72 z / z&lt;sub&gt;R&lt;/sub&gt;</td>
</tr>
</tbody>
</table>

Normalised Lateral Pile\Soil Interaction Curves for Static and Cyclic Loading

Figure 2.24 - Lateral Pile/soil Interaction Models, Matlock (1970)

Table 2.10 – Suggested Values for ε<sub>C</sub>, Barltrop & Adams (1991)

<table>
<thead>
<tr>
<th>Clay Type</th>
<th>Recommended Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brittle or Sensitive Clays</td>
<td>0.005</td>
</tr>
<tr>
<td>Disturbed, Remoulded or Unconsolidated Clays</td>
<td>0.02</td>
</tr>
<tr>
<td>All Other Clays</td>
<td>0.01</td>
</tr>
</tbody>
</table>
2.8.4 Rate Effects

The effect of the rate of lateral pipe/soil interaction in a sandy soil was examined by Hsu (1993). Hsu showed that the drag force on a pipe was proportional to a power of the pull-out velocity divided by the pipe diameter. The pipe/soil interaction curve used is based upon the hyperbolic function for sand soils given by Audibert & Nyman (1977) and Audibert et al (1984) where the parameters a and b are used to account for the effect of pull-out velocity. The equations for this model are given below.

\[ F'' = \frac{Y''}{a + bY''} \]  

(2.63)

where

- \( F'' \) is the normalised force
- \( Y'' \) is the normalised displacement
- \( a, b \) are dimensionless constants relating to pull-out speed given below

\[ a = 0.29 \left( \frac{V}{D} \right)^{-0.052} \]  

(2.64)

\[ b = 0.71 \left( \frac{V}{D} \right)^{0.025} \]  

(2.65)

2.9 Axial Soil Resistance

Axial resistance between a SCR and the soil is generally modelled using coulomb friction. Values for the friction coefficient are taken from pile skin friction tests. The equation for axial pipe/soil resistance given by Barltrop & Adams (1991) is presented below:

\[ T = \mu_A x \]  

(2.66)

where

- \( T \) is the axial force
- \( \mu_A \) is the coefficient of axial friction
- \( x \) is the axial displacement

The coefficient of axial friction has been reported to vary between 0.3 and 1.0 for a clay soil. Typical values for axial friction coefficient for clay and sands given in
BS8010 (1993) are shown in Table 2.11. Work conducted on submerged and drained soils suggests the following relationship for axial friction coefficient is used.

\[ \mu_A = \tan \phi' \]  

(2.67)

<table>
<thead>
<tr>
<th></th>
<th>Min</th>
<th>Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-cohesive (e.g. sand)</td>
<td>0.55</td>
<td>1.2</td>
</tr>
<tr>
<td>Cohesive (e.g. clay, silt)</td>
<td>0.3</td>
<td>1.0</td>
</tr>
</tbody>
</table>

The axial resistance of buried pipelines was discussed by Audibert et al (1984). He suggests that the following formula is used to relate the ultimate axial force, \( t_U \), to the ultimate axial displacement, \( x_U \), using the force/displacement model outlined in Figure 2.15.

\[ t_U = \pi D \mu_A S_U \]  

(2.68)

\[ x_u = 0.2 \text{ to } 0.4 \text{ inches} \]  

(2.69)

This method for calculating the axial resistance force was also given by Chaudhury (2001) who noted that for SCR analysis the contribution from axial soil resistance is negligible and can be ignored.

2.10 Vertical Upwards Pipe/Soil Interaction

Pipe laying contractors have reported high uplift resistance forces on pipelines during pipeline retrieval from a clay seabed. Pipelines are generally set down during storms to prevent them from buckling and being over-stressed. Once the storm has passed the pipeline is retrieved allowing the pipe laying operation to continue. However during the retrieval operation a force greater than the pipe weight is required to lift the pipe from the seabed. The greater pull-out force is due to pipe/soil suction or ‘mud suction’ as it is sometimes referred to (Foda, 1983). It was suggested by Yen et al (1975) that excessive high forces during pick-up operations could cause pipelines and SCRs to buckle.
Uplift resistance for a buried pipeline was examined by Audibert *et al* (1984) who stated that it can be considered as a reverse bearing capacity problem. The equation that represents this force and the mobilisation distance is given below. This model does not account for pipe/soil suction, but rather the soil weight and compaction of the backfill on top of the pipe. In addition this model was developed from static pipe/soil interaction tests.

\[ q_u = S_u N_{cv} D \]  
\[(2.70)\]

where

- \( N_{cv} \) is the vertical bearing capacity factor
- \( z_u = [0.1 \rightarrow 0.2] \times z \)  
\[(2.71)\]

Experiments conducted by Muga (1968), Vesic (1971), Byrne & Finn (1978) and Foda (1983) show that a suction force exists between a structure and a clay soil.

Muga (1968) examined the soil suction force mobilised when a vessel is raised from soft clay seabeds. The experiments, located in the San Francisco Bay, USA, used a range of test shapes including a concrete cylinder 1.37m (54") in diameter, 5.37m (112.5") long and a submerged weight of 9866kg (21,750lb). This was placed into the seabed from a platform, a constant force applied and the time taken for the object to break out from the soil recorded. These experiments showed that the structure/soil tension bond could be large and that the soil suction force reduced as a constant load was applied. Muga (1968) developed an empirical equation to model this:

\[ F = N A q_u e^{-R(t-t_0)} \]  
\[(2.72)\]

where

- \( F \) is the applied force
- \( N \) is a constant, reported to be 0.2 for the San Francisco Bay
- \( A \) is the horizontal projection of contact area
- \( R \) is a constant with units \( m^{-1} \), found to be 0.0054 for the San Francisco Bay
- \( t \) is the length of time to break-out
- \( t_0 \) is reference time interval
Further experiments conducted by Vesic (1971) showed that the structure/soil suction force is proportional to the soil shear strength and comparable to bearing capacity.

\[ q_{ult} = NS_U \]  

(2.73)

where

- \( q_{ult} \) is the ultimate suction pressure
- \( N \) is a constant – reported by Byrne & Finn (1978) as between 6.7 and 7.4 for rapid pull-out rate (0.126 mm/s) and by Muga (1968) as 0.2 for a slow pull-out speed
- \( S_U \) is the undrained soil shear strength

Byrne & Finn (1978) hypothesised that the suction force is transmitted by a change in pore-water pressure. They showed using triaxial test apparatus representing a skirted plate foundation that the breakout force, the force required to break the suction bond, and the time till break out are both affected by the pullout rate. Examination of the results given by Byrne & Finn (1978) suggests that the normalised break out force is 1.7 at a rate of 0.0038 mm/s, taking approximately 650 seconds to break out. The normalised break out force increases to 6.38 with a pull-out rate of 0.063 mm/s and takes approximately 30 seconds to break out.

Foda (1983) presented a theoretical approach to objects ‘breaking out’ from the seabed. The objectives were to determine the time to break-out when a constant force applied to the object on the sea bottom, simulating a salvage operation. He proposed an empirical relationship between the force applied and the time taken for a flat plate to break-out from the seabed, as shown in the equation below.

\[ t_b = \gamma S_U F^{-1.5} \]  

(2.74)

where

- \( t_b \) breakout time (s)
- \( \gamma \) constant which depends on the shape of the plate (N\(^{0.5}\) s m\(^2\))
- \( F \) pull-out force (N)
The research shows that the general theory relating to breakout resistance of an object on a seabed is limited to the application of salvage operations, where only an estimation of the maximum breakout force and the time to break out is required. However Bostrom et al (1998) conducted a limited test program to examine the effects of pipe/soil suction on subsea risers. The tests had a pipe section rigidly attached to an actuator that could be driven into and subsequently pulled out of a clay soil sample. An example of the results obtained is given in Figure 2.25 and shows the force/displacement curve for a pipe pushed into then pulled out of a clay soil. These tests along with reports from early phases of the STRIDE JIP sparked off the CARISIMA JIP testing program.

![Figure 2.25 - Pipe/Soil interaction Curve, Bostrom et al (1998)](image)

Bostrom et al (1998) created a numerical model based on the limited test data to assess the effect of pipe/soil suction on SCRs. This model showed a 37% increase in the peak bending moment at the TDP due to pipe/soil suction during quasi-static loading, Figure 2.26. The dynamic effect of SCR forces, and hence the effect on fatigue life was not examined.
2.11 SCR Trenches

Observations from a number of deepwater Gulf of Mexico (GoM) platforms show that SCRs supported on soft clay seabeds lie in wide trenches many diameters deep. The trenches, which are formed during the life of the SCR, have been observed in all SCR developments. Trenches have also been observed in the SCR model tests reported by Grant et al (1999). These SCR model tests used a 38.1mm diameter aluminium pipe that was 377m long and in 235m water depth. A photograph of the SCR trench mouth observed was taken by a remotely operated vehicle (ROV), Figure 2.27, and shows that the trench formed is approximately six diameters wide and four diameters deep.
2.11.1 Trench Formation

Most papers on the subject of trenching are concerned with burial of seabed pipelines rather than the trench formation around SCRs. Consequently the writer has examined other areas of pipe/soil interaction for possible SCR trenching mechanisms.

In experiments conducted by Lambrakos (1985) to examine the soil friction coefficients of a towed pipeline he discovered that small lateral oscillatory motions, which could be due to low wave forces or elastic friction forces, combined with a static load increases pipeline embedment. This is similar to the conclusions by Morris et al (1988), Verley & Sotberg (1994) and Verley & Lund (1995), where the latter two conducted analysis on the data collected during the PIPESTAB and AGA investigations.

Other trenching mechanisms that could dig, or help dig SCR trenches were proposed within early phases of the STRIDE JIP by 2H Offshore (1999c) and include:

- Wave loading on the vessel causing the SCR TDP to move to different locations. The SCR penetrates into the soil and a trench is formed as shown by 2H Offshore (1999b) and in Figure 2.4.
• Slow drift motions causing erosion of the soil near the trench mouth.
• As the riser moves towards the seabed large volumes of water are moved away from the TDZ and cause sediment transportation.
• Pumping of the riser on the seabed may produce large water velocities that dislodge and wash sediment away from the TDZ.
• Scour of the seabed around the riser by strong seabed currents.

At present there is no definitive evidence that supports or rules out any of the above trenching mechanism and more field observations are required to better understand this phenomena.

2.11.2 Trench Stability

A SCR trench can be thought of as a shallow excavated foundation and the stability of the trench wall estimated using slope stability equations. Using this analogy the depth of a vertical trench was shown by Yen et al (1975) and Craig (1992) to be limited by soil heave at the bottom of the trench. The maximum vertical trench wall height, H, can be conservatively calculated using the equation below.

\[ H = \frac{2S_u}{\gamma_s} \]  \hspace{1cm} (2.75)

The work conducted on slope stability by Taylor (1937) shows that less conservative values of slope height can be calculated using the following equation that incorporates stability coefficients, Ns, which for a trench with near vertical walls can be assumed to be 0.25.

\[ H = \frac{S_u}{N_s \gamma_s} \] \hspace{1cm} (2.76)

Using the very soft clay data from DNV (1998) given in Table 2.4 (Su = 5 MPa, \( \gamma = 4.4 \text{ kN/m}^3 \)) the maximum trench wall height is 4.5m.

Yen et al (1975) presented a trench stability equation that was based on sub-marine jet sled excavations. The equilibrium of the trench can be expressed in terms of the trench height, the pore water pressure and submerged soil weight as shown below.
\[ H = \frac{2SG}{(\gamma_s \cos \alpha + n)\sin \alpha} \]  
\[ (2.77) \]

where
- \( H \) is the height of the trench
- \( \alpha \) is the slope at which wall slumping is expected to progress
- \( n \) is a parameter reflecting the rate of excess pore pressure change with depth. For a normally consolidated clay it can be calculated using the equation below:

\[ n = A_fk_a\gamma_s \]  
\[ (2.78) \]

where
- \( A_f \) is the soil pore pressure at failure
- \( k_a \) is the coefficient of active lateral pressure

2.11.3 Trench Models

Existing trench models used in SCR analysis assume a conservative trench profile that is steep sided, approximately three diameters deep and one diameter wide as suggested by Thethi & Moros (2001) and shown in Figure 2.28 below. The trench model given by Thethi & Moros (2001) was based on observations from ROV trench surveys collected and work conducted during the STRIDE JIP.

![Figure 2.28 - Conservative Trench Model Used in SCR Analysis](image)

2.12 Summary

Steel catenary risers are elegantly simple in conception, being a steel pipe draped from a floating production unit that slope gently towards the seabed. The catenary zone of a SCR can be analysed statically using the catenary equations or statically and dynamically using finite element techniques. However it is in the TDZ, where
the SCR interacts with the seabed, that our current knowledge and analysis techniques are limited. Current riser design may not model the TDZ appropriately as analysis is conducted using simplified pipe/soil interaction models and does not account for SCR trenches or pipe/soil suction forces.

From the research conducted it can be seen that in deepwater environments, where SCRs are installed, the seabed tends to be made from saturated, high plasticity soft clays. The published data indicates that the undrained soil shear strength and plasticity index are the most important parameters for pipe/soil interaction in clay soils. However the undrained soil shear strength has been shown by Taylor (1943), Brennholden & Stokkeeland (1992) and Dunlap et al (1990), among others to be dependant on interaction rate, consolidation (or rest) time and load and hysteresis (or number of previous cycles). Most of the pipe/soil interaction models researched are derived from bearing capacity formulation and do not account for rate, consolidation or hysteresis. Consequently they are applicable for static analysis or high frequency earthquake loading, but may not be appropriate for wave frequency dynamic analysis. Dynamic pipe/soil interaction models do exist, however there is a wide range of dynamic soil stiffnesses that have been proposed, ranging from 0.88 times to 100 times the static soil stiffness. In addition the existing models are predominantly based on data for buried pipelines or piles where the dynamic motions tend to be small, whereas in the TDZ the riser tends to rest within a trench on top of the seabed and the TDP can experience large in-line and transverse motions of up to ±8m and ±200m respectively and move vertically upwards out of the trench.

Observations from pipeline and SCR installation contractors note that when pipelines are lifted off the seabed they experience greater forces than expected and that these forces could cause the pipeline to buckle. This force, which has been identified as pipe/soil suction, is also assumed to exist within the TDZ of SCRs. Tests conducted by Bostrom et al (1998) showed that a pipe/soil suction bond does exist and that pipe/soil suction forces can increase the bending stress in the SCR at the TDP during static loading. However the effect of pipe/soil suction on SCR extreme stress and fatigue life during dynamic loading is still unknown.
Trenches are a common feature of the seabed where SCRs are present. However there is little published literature that details the trench shape, and even less suggesting how to model them. Consequently the implications of trenches on SCR stress distribution and fatigue life is unknown.

2.12.1 Summary of References

The references used for the riser analysis, subsea soils and pipe/soil interaction are classified in Table 2.12. A description of the columns and a list of the symbols used are given below. Note that if any of the designations given are not applicable the cell will contain a "-" symbol.

Area – the area of relevance for the reference

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>foundations</td>
</tr>
<tr>
<td>O</td>
<td>offshore structures</td>
</tr>
<tr>
<td>PS</td>
<td>pipe/soil interaction</td>
</tr>
<tr>
<td>R</td>
<td>risers</td>
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<tr>
<td>S</td>
<td>soils in general</td>
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<td>SS</td>
<td>subsea soils</td>
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</table>

Type – describes the content of the reference, whether testing, theory or both

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<td>F</td>
<td>full scale model tests</td>
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<td>M</td>
<td>field measurements</td>
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<tr>
<td>T</td>
<td>theory</td>
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System – type of riser/model/pipeline described in reference

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
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<tr>
<td>P</td>
<td>pipeline</td>
</tr>
<tr>
<td>PF</td>
<td>pile foundation</td>
</tr>
<tr>
<td>R</td>
<td>all riser types</td>
</tr>
<tr>
<td>SCR</td>
<td>steel catenary riser</td>
</tr>
<tr>
<td>TDP</td>
<td>the paper focuses specifically on the touchdown zone</td>
</tr>
<tr>
<td>TTR</td>
<td>top tensioned riser</td>
</tr>
<tr>
<td>O</td>
<td>other system</td>
</tr>
</tbody>
</table>

Depth – water depth in metres of riser study or model, or if general

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>on dry land</td>
</tr>
<tr>
<td>S</td>
<td>shallow depths, less than 500m</td>
</tr>
<tr>
<td>M</td>
<td>mid depth, between 500m and 1000m</td>
</tr>
<tr>
<td>D</td>
<td>deep, between 1000m and 2000m</td>
</tr>
<tr>
<td>U</td>
<td>ultra deep, greater than 2000m</td>
</tr>
</tbody>
</table>

FE Code – the name of the finite element analysis code used, if applicable

<table>
<thead>
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<th>Code Name</th>
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<tbody>
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<td>AB</td>
<td>ABAQUS, HKS (2004)</td>
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<tr>
<td>F</td>
<td>FLEXCOM, MCS (2004)</td>
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<tr>
<td>O</td>
<td>ORCAFLEx, Orcina Ltd (2004)</td>
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<tr>
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<td>RIFLEX, Marintek (1997)</td>
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<td>M</td>
<td>the FE method rather than a specific code</td>
</tr>
<tr>
<td>H</td>
<td>hand calculations</td>
</tr>
</tbody>
</table>

P/S – the type of pipe/soil model or interaction described

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
</tr>
</thead>
<tbody>
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Comments – any additional comments and/or useful information not covered by the other columns.

Relevance – the reference is rated on the relevance to this thesis from 1 (not very) to 5 (extremely).
Table 2.12 – Summary of References for Riser Analysis, Subsea Soils and Pipe/Soil Interaction

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<td>H</td>
<td>V</td>
<td>C</td>
<td>Slope stability coefficients for trench walls</td>
<td>3</td>
</tr>
<tr>
<td>Thethi &amp; Moros (2001)</td>
<td>PS</td>
<td>T</td>
<td>SCR</td>
<td>M, D, U</td>
<td>A, L, V, S</td>
<td>C</td>
<td>Presents the TDP Location envelope</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>True (1974)</td>
<td>PS</td>
<td>S</td>
<td>P</td>
<td>-</td>
<td>-</td>
<td>V</td>
<td>C</td>
<td>Model tests of projectiles into seabed soils</td>
<td>3</td>
</tr>
<tr>
<td>Vas &amp; Patel (2000)</td>
<td>R</td>
<td>T</td>
<td>R</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Elastic marine cables in sheared currents</td>
<td>4</td>
</tr>
<tr>
<td>Verley &amp; Soberg (1994)</td>
<td>PS</td>
<td>S</td>
<td>P</td>
<td>-</td>
<td>-</td>
<td>L</td>
<td>S</td>
<td>Empirical solutions</td>
<td>3</td>
</tr>
<tr>
<td>Vesic (1971)</td>
<td>PS</td>
<td>S, T</td>
<td>P</td>
<td>-</td>
<td>-</td>
<td>S</td>
<td>C</td>
<td>Assessment of pipe/soil suction force</td>
<td>5</td>
</tr>
<tr>
<td>Yen et al (1975)</td>
<td>SS</td>
<td>M</td>
<td>F</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>C</td>
<td>Deepwater geotechnical data</td>
<td>4</td>
</tr>
</tbody>
</table>
3.0 SCR TRENCH SURVEYS AND ANALYSIS

3.1 Introduction

Knowledge of the shape of a seabed trench, due to the presence of a steel catenary riser (SCR), can improve the accuracy of stress and fatigue damage calculations in SCR analysis. In finite element analysis models the seabed is considered to be flat, which has been shown by 2H Offshore (2002c) to generate conservative stress distributions, with the peak stress at the touchdown point (TDP). If the trench is shaped around the SCR in the touchdown zone (TDZ), then the length of riser over which the peak stress is distributed at the TDP will be larger, reducing the peak stress, and hence the fatigue damage will be lower than expected.

The shapes of the trenches occurring in practice are obtained by examining remotely operated vehicle (ROV) surveys of the TDZ of existing SCRs. Currently there are few detailed studies of SCR trenches in the public domain; however, during the STRIDE JIP the opportunity arose to video the trench surrounding the TDP of the Allegheny SCR in the Gulf of Mexico (GoM). The video showed the shape and extent of the trench as the ROV was manoeuvred along the length of the TDZ. Consequently efforts were made to collect other SCR trench videos. A total of six trenches were included on the STRIDE JIP CD-ROM with a brief technical note describing the location and general shape of the riser trench. A summary of these SCR trench surveys is given in Table 3.1. All videos are used with the permission of STRIDE, except the Auger trench video that is used with permission from Shell.

The surveys are presented as still photographs taken from the SCR trench videos. Analysis has been conducted to determine the shape and extent of the SCR trenches. Comparisons between the trenches are used to show common features that can be used in SCR analysis to more accurately calculate riser stress and fatigue life.
Table 3.1 – Summary of Video Surveyed Risers

<table>
<thead>
<tr>
<th>Field</th>
<th>Vessel</th>
<th>Operator</th>
<th>Water Depth</th>
<th>Riser</th>
<th>Time Since Installed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allegheny, Green Canyon, GoM</td>
<td>Atlantia</td>
<td>British Borneo /</td>
<td>992m</td>
<td>12-3/4&quot; Oil Export</td>
<td>7 months</td>
</tr>
<tr>
<td></td>
<td>Seastar mini</td>
<td>Agip</td>
<td></td>
<td>12-3/4&quot; Gas Export</td>
<td>16 months</td>
</tr>
<tr>
<td></td>
<td>TLP</td>
<td></td>
<td>7&quot; Production</td>
<td></td>
<td>16 months</td>
</tr>
<tr>
<td>Marlin, GoM</td>
<td>TLP</td>
<td>BP</td>
<td>988m</td>
<td>14&quot; Gas Export</td>
<td>1 year</td>
</tr>
<tr>
<td>P18, Marlim, Compos Basin,</td>
<td>P18 Semi-</td>
<td>Petrobras</td>
<td>900m</td>
<td>10&quot; Gas Export</td>
<td>2 years</td>
</tr>
<tr>
<td>Brazil</td>
<td>submersible</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Auger, GoM*</td>
<td>Catenary</td>
<td>Shell</td>
<td>872m</td>
<td>12-3/4&quot; Export</td>
<td>4 years</td>
</tr>
<tr>
<td></td>
<td>Moored TLP</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The Auger Trench video is used with the permission of Shell. Further distribution or external use this video will require further review and written permission by Shell.

3.2 Assessment of Trench Surveys

The trench surveys were conducted using an ROV that was piloted along the TDP region of the SCR. The information from the onboard camera was recorded onto videotape then digitalised into the MPG (Moving Picture Experts Group) format. The writer has reviewed the videos and compiled tables that detail the video elapsed time (VET), the width, depth and features of the trenches. In addition snapshots are taken from the digitalised film to illustrate features of the trenches. The trench and riser are not always clear in the video or snapshots as they can be obscured by particles suspended in the water and sediment washed up in front of the camera by the ROV’s propulsion system. Important features are picked out on the photographs using the following key:

- The location of the riser pipe on the photographs is highlighted using a pair of dotted red lines,
- The cross-section of the trench is picked out with a black line,
- The lip of the trench by a dotted white line,
• Backfill that covers the pipe is highlighted in orange.

Measurements of the riser trenches are taken from the video and snapshots using the following methods:

• The width and depth of the trench are extrapolated by scaling them with the diameter of the SCR. All dimensions are approximate and given in terms riser diameter.
• The length and direction are determined from the ROV speed and heading if they are displayed along the top of the video.
• If the riser is obscured then, if possible, the width and depth of the trench are estimated using the position of the ROV relative to the trench.

The terminology used to describe the trenches is defined below and shown graphically in Figure 3.1 and Figure 3.2.

• Catenary, buried and surface zones are those defined in the literature review and given in Figure 3.1. In the observation tables this is indicated in the ‘zone’ column by a C, B or S for Catenary, Buried or Surface respectively.
• Soil covering the riser is called backfill. In the observation tables this is indicated in the ‘Backfill’ column by an N, B or I for None, Backfill or Intermittent respectively.
• The location of each photograph is given on a trench schematic similar to Figure 3.1.
• The orientation of the riser in the photos is defined as either towards the vessel or towards the flowline as defined in Figure 3.1.
• Trench floor is the lowest section of the trench as shown in Figure 3.2.
• Trench depth is measured from the seabed to the trench floor, ignoring any additional pipe embedment.
• The embedment depth is the depth from the bottom of the trench to the bottom of the pipe.
• Trench lip is the end of the trench wall that connects to the seabed surface.
• Trench walls are described as being vertical, steep (near vertical) or sloped. The gradient of the slope is given by the angle to the horizontal as shown in Figure 3.2.

![Figure 3.1 - Overview of Trench Definitions and Assumed Shape](image)

**Catenary Zone**
the riser is free hanging between the vessel and the TDP

**Buried Zone**
the riser is within the trench, interacting with the soil

**Surface Zone**
the riser rests on the seabed

![Figure 3.2 - Definition of Trench Dimensions](image)

3.3 Analysis of Trench Surveys

3.3.1 Allegheny, Green Canyon, Gulf of Mexico

The Allegheny Tension Leg Platform (TLP) is in Green Canyon, Block 254 in the Gulf of Mexico, USA as shown in Figure 3.3. The field was brought online in
September 1999 and is in 992m water depth. The platform uses two 12.75" (0.324m) outer diameter SCRs for oil and gas export and two 7" (0.178m) outer diameter SCRs for production.

![Figure 3.3 – Location of Allegheny Field](image)

The opportunity to film the TDP arose when the data from the STRIDE standalone accelerometer data logger pods were being retrieved with a ROV in February 2000, seven months after the installation of the riser. The ROV was piloted along the TDZ of both the oil and gas export SCRs, however the footage of the gas export SCR was obscured by sediment kicked up by previous ROV operations. The surveys started at the catenary zone then flew above the buried zone towards the surface zone. Nine months later, when the data loggers were retrieved, fly-bys of the gas and the oil export SCRs and the production SCRs was conducted. Unfortunately, the footage of the oil export riser trench was not recorded, however, it was noted by those observing the survey footage on the vessel that the oil and gas export SCR trenches appeared to be similar.

The observations on the Allegheny videos for the gas export riser seven months after installation, the oil export riser 16 months after installation and the production riser 16 months after installation are given below.
Allegheny Trench Survey, Gas Export SCR, Seven Months After Installation

A description of the SCR trench video is given below with a summary of the trench width, depth and features in Table 3.2. Photographs taken from the video are given in Figure 3.4 with a summary sketch of the SCR trench showing the locations of the photographs with approximate dimensions of the trench wall given in Figure 3.5.

The video shows the trench as the ROV is piloted along the riser, starting at the TDP and flying towards the flowline at an estimated speed of 0.3m/s. The first frames of the trench video show the SCR in a wide and shallow trench near the TDP that is illustrated in Figure 3.4, picture A. After 4s VET (approximately 1.2m along the trench), the video cuts to a steep sided trench in the buried zone, where the riser is covered with backfill as shown in picture B. As the ROV continues to fly towards the flowline, the trench is observed to reduce in width and depth (pictures C to E) until 1:44 VET, where the trench has tapered away so that the riser is embedded in the seabed as shown in picture F. This is considered to be the start of the surface zone. Fissures, or tension cracks are seen in the seabed running parallel to the trench in pictures C, D and E, and end at 1:40 VET. It is observed that the trench is narrower where there are tension cracks in the seabed but no backfill covering the riser, indicating that the backfill comes from a collapsed trench wall.
Table 3.2 – Observations of Allegheny Gas Export Trench

<table>
<thead>
<tr>
<th>VET (min:s)</th>
<th>Trench Depth (D)</th>
<th>Trench Width (D)</th>
<th>Picture</th>
<th>Zone</th>
<th>Backfill</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0</td>
<td>2</td>
<td>10</td>
<td>A</td>
<td>C</td>
<td>N</td>
<td>Trench mouth.</td>
</tr>
<tr>
<td>5.1</td>
<td>5</td>
<td>5</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>Steep sided trench with tension cracks on seabed. Riser not visible through backfill.</td>
</tr>
<tr>
<td>20.0</td>
<td>4.5</td>
<td>4.5</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>35.0</td>
<td>4.0</td>
<td>4.5</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>40.0</td>
<td>4.0</td>
<td>4.0</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>42.0</td>
<td>4.0</td>
<td>3.0</td>
<td>C</td>
<td>B</td>
<td>I</td>
<td></td>
</tr>
<tr>
<td>48.0</td>
<td>3.5</td>
<td>3.0</td>
<td>B</td>
<td>I</td>
<td></td>
<td></td>
</tr>
<tr>
<td>51.3</td>
<td>2.0</td>
<td>2.5</td>
<td>B</td>
<td>I</td>
<td></td>
<td></td>
</tr>
<tr>
<td>57.1</td>
<td>2.0</td>
<td>2.5</td>
<td>D</td>
<td>B</td>
<td>I</td>
<td></td>
</tr>
<tr>
<td>1:05.0</td>
<td>1.3</td>
<td>2.2</td>
<td>B</td>
<td>I</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:14.0</td>
<td>1.0</td>
<td>2.0</td>
<td>B</td>
<td>N</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:26.0</td>
<td>0.5</td>
<td>2.0</td>
<td>E</td>
<td>B</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>1:38.0</td>
<td>0.3</td>
<td>1.5</td>
<td>B</td>
<td>N</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:42.0</td>
<td>0.1</td>
<td>1.3</td>
<td>B</td>
<td>N</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:44.1</td>
<td>0.0</td>
<td>1.0</td>
<td>S</td>
<td>N</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:56.0</td>
<td>0.0</td>
<td>1.0</td>
<td>F</td>
<td>S</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No observed trench.</td>
</tr>
</tbody>
</table>
Figure 3.4 – Allegheny Trench, Seven Months After Installation, Pictures A – F
Allegheny Trench Survey, Oil Export SCR, 16 Months After Installation

A description of the SCR trench video is given below with a summary of the trench width, depth and features in Table 3.3. Photographs taken from the video are given in Figure 3.7 with a summary sketch of the SCR trench showing the locations of the photographs with approximate dimensions of the trench wall in Figure 3.6.

The video shows the trench as the ROV is piloted along the riser, starting at the TDP and flying towards the flowline at an estimated speed of 0.35m/s. In the first 10s a cloud of sediment that has been thrown up by the ROV thrusters obscures the riser and trench. After the ROV has flown through the cloud the vertical left hand trench wall near the TDP becomes visible as shown in Figure 3.7, picture A. The riser can be seen suspended above the trench for the next 17s, then disappears beneath backfill. The trench width reduces from 4.0D to 1.0D over the next minute of video as shown in pictures B, C and D. The trench depth is obscured by sediment that is suspended in the water and can only be estimated using the ROV until 50.0s VET at which point it is 2.5D deep and 3.0D wide as shown in picture B. As the ROV continues to fly towards the flowline the riser can clearly be seen in the trench until 1:27 VET where the trench tapers off and the riser is considered to be in the surface zone. At 1:36 VET the ROV comes to rest on top of the riser.
Table 3.3 – Observations of Allegheny Oil Export Trench

<table>
<thead>
<tr>
<th>VET (min:s)</th>
<th>Trench Depth (D)</th>
<th>Trench Width (D)</th>
<th>Picture</th>
<th>Zone</th>
<th>Backfill</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Trench and riser obscured by sediment in water.</td>
</tr>
<tr>
<td>20.0</td>
<td>1.6</td>
<td>5.0</td>
<td>A</td>
<td>C</td>
<td>N</td>
<td>Riser faintly visible through sediment in water.</td>
</tr>
<tr>
<td>27.0</td>
<td>1.6</td>
<td>4.0</td>
<td>C</td>
<td>N</td>
<td></td>
<td>The trench floor and riser are hidden by sediment suspended in the water.</td>
</tr>
<tr>
<td>40.0</td>
<td>2.5</td>
<td>3.5</td>
<td>B</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50.0</td>
<td>2.5</td>
<td>3.0</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>57.0</td>
<td>3.0</td>
<td>3.5</td>
<td>B</td>
<td>N</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:05.0</td>
<td>2.5</td>
<td>2.5</td>
<td>C</td>
<td>B</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>1:12.0</td>
<td>1.0</td>
<td>2.0</td>
<td>B</td>
<td>N</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:21.0</td>
<td>0.5</td>
<td>1.5</td>
<td>D</td>
<td>B</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>1:27.0</td>
<td>0.0</td>
<td>1.0</td>
<td>S</td>
<td>N</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:32.2</td>
<td>0.0</td>
<td>1.0</td>
<td>E</td>
<td>S</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>1:36.0</td>
<td>0.0</td>
<td>1.0</td>
<td>S</td>
<td>N</td>
<td></td>
<td>ROV rest on top of riser.</td>
</tr>
</tbody>
</table>

Figure 3.6 – Sketch of Allegheny Trench 16 Months After Installation
Figure 3.7 – Allegheny Trench, 16 Months After Installation, Pictures A – E

Allegheny Production SCRs, 16 Months After Installation
This video shows ROV surveys of two of the 7" production SCRs attached to the Allegheny platform. The video consists of three ROV fly-bys of two risers. The first two fly-bys cover the same trench, initially starting at the surface zone and flying towards the catenary zone. The second starts in the catenary zone and flies towards the TDP, where it stops. The 3rd ROV sweep is of a different trench and flies from the surface zone towards the vessel.
A description of the first trench surveyed (denoted Trench A) is given below with a summary of the trench width, depth and features in Table 3.4. Photographs of this trench taken from the video are given in Figure 3.11 with a summary sketch of the SCR trench showing the locations of the photographs with approximate dimensions of the trench wall in Figure 3.8.

The first ROV sweep of Trench A starts in the surface zone and travels along the riser towards the vessel at an estimated speed of 0.2m/s. The first 8s of video show the riser in the surface zone and the start of the buried zone. The video is then cut by approximately two minutes and at 8s VET the survey resumes from a similar but different position in the buried zone that is shown in Figure 3.9, picture A. The trench at this time has vertical walls that are approximately 1.5D tall and 4.0D apart. As the ROV flies along the riser, the trench widens and deepens. At 16s VET, a second trench is observed to have begun to form within the existing trench and is shown in picture B, Figure 3.9. The internal trench sits against the right hand side wall of the outer trench, which is vertical. The left hand side wall of the inner trench is sloped while the left hand side wall of the outer trench is vertical. As the ROV continues along the riser, the internal trench becomes wider and deeper, and at 30s VET the outer trench is 5D deep and over 7D wide while the inner trench is 2D deep and 4.5D wide as shown in picture C. At 42s the inner and outer trenches merge into one trench that is 5D deep and 10D wide. This trench shape continues for 4s VET where it is observed that the riser is once again sitting in a small trench within a larger one as shown in picture D. At 44.8s the riser loses contact with the seabed in the inner trench and catenary zone is determined to have begun. At 58s VET the video of this ROV sweep ends.

The second ROV sweep of trench A starts at 58.0s with the riser suspended in the catenary zone. As the ROV flies towards the TDP the start of the trench mouth in the catenary zone comes into view as an indent in the seabed as shown in picture E. The riser sits within the trench mouth in a smaller indent that is 3D wide and 1D deep. The riser then carries along the trench towards the flowline. The trench deepens to 3D while the width reduces to 14D at 1:15.5 VET where the video of this riser ends.
The Allegheny production riser trench B survey starts at 1:15.5 VET and shows the riser in the buried zone near the flowline with the ROV pointing towards the vessel. A summary of the trench depth, width and features is given in Table 3.5. Photographs taken from the video are given in Figure 3.11 with a sketch showing the approximate location of the photos in Figure 3.10.

The Allegheny Production SCR Trench B riser, shown in Figure 3.11, picture A is sitting in a trench within a trench where both trenches share the left hand wall, which is sloped at 45° to the horizontal. The inner trench right hand wall is almost vertical and the inner trench lips are 6D apart. The outer trench walls are sloped and estimated to be 2D high and 9D apart. As the ROV flies towards the vessel the riser cuts into the right hand wall of the inner trench so that after 5S VET the right hand inner trench wall is vertical. Further along the trench the riser has eroded more soil away from the right hand inner trench wall so that the soil overhangs the pipe as shown in picture B. The left hand wall of the inner trench is vertical and the trench width and depth are both 1.5D. The riser continues to sit in the inner trench, which gradually gets deeper and wider, so that at 1:28.5 VET the inner trench is 2D deep and 2D wide. The outer trench has not increased in width or depth. At 1:34 VET soil is observed to surround the riser and at 1:39 the riser is completely covered by backfill, as shown in picture D. At 1:56.8VET the riser emerges from the backfill in a 6D wide and 3D deep trench as shown in picture E. The riser then continues in an inner trench that is 3D wide and 1D deep in the outer trench that is approximately 3D deep and over 5D wide. At 1:05.7 VET the riser enters the catenary zone and at 2:11.1 the trench video ends.

<table>
<thead>
<tr>
<th>VET (min:s)</th>
<th>Trench Depth (D)</th>
<th>Trench Width (D)</th>
<th>Picture</th>
<th>Zone</th>
<th>Backfill</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.2</td>
<td>0.3</td>
<td>1.7</td>
<td>B</td>
<td>N</td>
<td></td>
<td>Shallow trench with 45° sloped walls</td>
</tr>
<tr>
<td>8.5</td>
<td>1.0</td>
<td>3.5</td>
<td>A</td>
<td>B</td>
<td>I</td>
<td>Video cuts to different section of trench</td>
</tr>
</tbody>
</table>

Table 3.4 – Observations of Allegheny Production SCR Trench A
<table>
<thead>
<tr>
<th>VET (min:s)</th>
<th>Trench Depth (D)</th>
<th>Trench Width (D)</th>
<th>Picture</th>
<th>Zone</th>
<th>Backfill</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>16.5</td>
<td>2.5, 0.2</td>
<td>4.5, 1.2</td>
<td>B</td>
<td>B</td>
<td>N</td>
<td>The riser sits in a small trench within a larger outer trench. Both trenches have steep walls</td>
</tr>
<tr>
<td>21.7</td>
<td>4.0, 1.0</td>
<td>5.5, 2.0</td>
<td>B</td>
<td></td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>30.0</td>
<td>5.0, 2.0</td>
<td>7.0, 4.5</td>
<td>C</td>
<td>B</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>34.5</td>
<td>5.0, 2.0</td>
<td>&gt;7.0, 5.0</td>
<td>B</td>
<td>B</td>
<td></td>
<td></td>
</tr>
<tr>
<td>42.5</td>
<td>5.0</td>
<td>10.0</td>
<td>B</td>
<td>B</td>
<td></td>
<td>Inner and outer trenches join together</td>
</tr>
<tr>
<td>44.8</td>
<td>5.0</td>
<td>10.0</td>
<td>B</td>
<td></td>
<td>N</td>
<td>Start of trench mouth</td>
</tr>
<tr>
<td>47.5</td>
<td>2.0, 1.0</td>
<td>14.0, 3.0</td>
<td>D</td>
<td>B</td>
<td>N</td>
<td>Riser sits in an indent within trench mouth</td>
</tr>
<tr>
<td>54.0</td>
<td>1.0, 1.0</td>
<td>15.0, 3.0</td>
<td>C</td>
<td></td>
<td>N</td>
<td>End of first ROV sweep</td>
</tr>
<tr>
<td>58.0</td>
<td>-</td>
<td>-</td>
<td>C</td>
<td></td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>1:00.0</td>
<td>0.5</td>
<td>6.0</td>
<td>E</td>
<td>C</td>
<td>N</td>
<td>Start of second ROV sweep</td>
</tr>
<tr>
<td>1:04.4</td>
<td>2.0</td>
<td>&gt;12.0</td>
<td>F</td>
<td>C</td>
<td>N</td>
<td>Riser sits in an indent within trench mouth</td>
</tr>
<tr>
<td>1:06.9</td>
<td>2.0</td>
<td>15.0</td>
<td>C</td>
<td></td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>1:15.5</td>
<td>3.0</td>
<td>14.0</td>
<td>B</td>
<td></td>
<td>N</td>
<td>End of second ROV sweep</td>
</tr>
</tbody>
</table>

Figure 3.8 – Sketch of Allegheny Production Riser Trench A
Figure 3.9 – Allegheny Production Riser A, Pictures A – F
### Table 3.5 – Observations of Allegheny Production Riser Trench B

<table>
<thead>
<tr>
<th>VET (min:s)</th>
<th>Trench Depth (D)</th>
<th>Trench Width (D)</th>
<th>Picture</th>
<th>Zone</th>
<th>Backfill</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:15.5</td>
<td>1.5</td>
<td>6.0</td>
<td>A</td>
<td>B</td>
<td>N</td>
<td>Buried zone</td>
</tr>
<tr>
<td>1:22.0</td>
<td>4.0, 1.5</td>
<td>10.0, 1.5</td>
<td>B</td>
<td>B</td>
<td>N</td>
<td>Trench within a trench. Right hand trench wall is vertical and overhangs the riser</td>
</tr>
<tr>
<td>1:28.5</td>
<td>&gt;3.0, 2.0</td>
<td>&gt;5.0, 2.0</td>
<td>C</td>
<td>B</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>1:34.0</td>
<td>&gt;3.0, 2.0</td>
<td>&gt;5.0, 3.0</td>
<td>B</td>
<td>B</td>
<td></td>
<td>Soil surrounds the riser</td>
</tr>
<tr>
<td>1:46.8</td>
<td>3.0</td>
<td>5.0</td>
<td>D</td>
<td>B</td>
<td>B</td>
<td>Riser completely covered by soil</td>
</tr>
<tr>
<td>1:57.5</td>
<td>3.0</td>
<td>5.0</td>
<td>E</td>
<td>C</td>
<td>B</td>
<td>Riser emerges from the backfill</td>
</tr>
<tr>
<td>2:05.7</td>
<td>3.0</td>
<td>5.0</td>
<td>C</td>
<td>N</td>
<td></td>
<td>Riser enters catenary zone</td>
</tr>
<tr>
<td>2:11.1</td>
<td>1.5</td>
<td>6.0</td>
<td>F</td>
<td>C</td>
<td>N</td>
<td>Catenary zone</td>
</tr>
</tbody>
</table>

---

**Figure 3.10 – Sketch of Allegheny Production Riser Trench B**
3.3.2 Marlin

The Marlin development is located in Viosca Knoll Block 915, approximately 125 miles southeast of New Orleans in the Gulf of Mexico, as illustrated by Figure 3.12. Marlin is 25% owned by Shell and 75% by BP, who operate the field. The SCR is a 14” Gas Export line, connected to a Tension Leg Platform (TLP) in 988m water depth. The video was taken as part of routine survey work.
A description of the SCR trench video is given below with a summary of the trench width, depth and features in Table 3.6. Photographs taken from the video are given in Figure 3.14 with a summary sketch of the SCR trench showing the locations of the photographs with approximate dimensions of the trench wall in Figure 3.13.

The video starts with the ROV hovering above the buried zone looking towards the vessel as shown in Figure 3.14, picture A. The trench at this location is 2D deep, and the trench lips are 3D apart. The riser is lying next to the right hand trench wall that is vertical. The left hand trench wall is sloped at 45° with a step at 1D depth that is 0.5D wide. As the ROV flies along the riser the trench increases in width to 7D at 6.2s VET and 9D at 19.6s VET as shown in pictures B and C respectively. The trench depth increases to 3.5D over the same VET. The riser is observed to sit near to the middle of the trench in a small indent that is 0.5D deep and 2D wide within the trench. At 23.7s VET the riser enters the catenary zone, losing contact with the seabed. The ROV follows the riser a short distance and then turns 90° to face the pipe as shown in picture D. The video shows that at this location the riser moves vertically by 0.1D with a period of approximately 3s.

The video then cuts to show the surface zone of a second SCR trench in the Marlin field with the ROV pointing towards the vessel. The ROV travels along the riser showing the tail end of the trench forming. At 51.8s VET the trench is 0.125D wide
and 0.75D deep as shown in picture E. The trench develops further and at 57.8s, when the ROV stops flying along the riser the trench has steep sides that are 1.5D deep and 1.5D apart as shown in picture F. The ROV stops at this point to examine the two black ‘lumps’ that are in the trench on either side of the riser in picture F. Closer examination reveals that these are fish. At 1:08.5 VET the video records static, but restarts again at 1:11.7 VET with the ROV further along the trench in the buried zone pointing towards the vessel. The trench, which is shown in picture G, is observed to be steep sided, 2.5D deep and 3.5D wide with the riser sitting in a shallow indent in the trench floor. The ROV follows the riser towards the vessel and shows that the steep sided trench widens to 5D and deepens to 3.5D nine seconds VET later. The indent in the bottom of the trench also increases in size as shown in picture H and is 2D wide and 1D deep. At 1:27.0 VET the riser enters the catenary zone. The surrounding trench is steep sided, 4D deep and 7D wide while the indent is 2.5D wide and just over 1D deep. The ROV then turns 90° and recorded the riser moving vertically in the trench with amplitude of around 0.1D with a period of 6s.
Table 3.6 – Observations of Marlin SCR Trench

<table>
<thead>
<tr>
<th>VET (min:s)</th>
<th>Trench Depth (D)</th>
<th>Trench Width (D)</th>
<th>Picture</th>
<th>Zone</th>
<th>Backfill</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>2.0</td>
<td>3.0</td>
<td>A</td>
<td>B</td>
<td>N</td>
<td>Left hand side wall sloped, right hand wall steep</td>
</tr>
<tr>
<td>6.2</td>
<td>3.0</td>
<td>7.0</td>
<td>B</td>
<td>B</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>19.6</td>
<td>3.5</td>
<td>9.0</td>
<td>C</td>
<td>B</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>23.7</td>
<td>4.0</td>
<td>10.0</td>
<td>C</td>
<td>N</td>
<td></td>
<td>Start of catenary zone</td>
</tr>
<tr>
<td>29.8</td>
<td>3.0</td>
<td>10.0</td>
<td>D</td>
<td>C</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>50.8</td>
<td>3.0</td>
<td>10.0</td>
<td>C</td>
<td>N</td>
<td></td>
<td>End of first sweep</td>
</tr>
<tr>
<td>50.8</td>
<td>0.0</td>
<td>1.0</td>
<td>S</td>
<td>N</td>
<td></td>
<td>Start of second sweep</td>
</tr>
<tr>
<td>51.8</td>
<td>0.75</td>
<td>1.25</td>
<td>E</td>
<td>B</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>57.8</td>
<td>1.5</td>
<td>1.5</td>
<td>F</td>
<td>B</td>
<td>N</td>
<td>ROV stops travelling along the riser</td>
</tr>
<tr>
<td>1:08.5</td>
<td>1.5</td>
<td>1.5</td>
<td>B</td>
<td>N</td>
<td></td>
<td>Static in video for 2.3s, Video starts again at 1:11.7</td>
</tr>
<tr>
<td>1:11.7</td>
<td>2.5</td>
<td>3.5</td>
<td>G</td>
<td>B</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>1:20.3</td>
<td>3.5</td>
<td>5.0</td>
<td>H</td>
<td>B</td>
<td>N</td>
<td>Steep sided trench with riser in an indent in the middle</td>
</tr>
<tr>
<td>1:27.0</td>
<td>4.0</td>
<td>5.5</td>
<td>B</td>
<td>N</td>
<td></td>
<td>Riser enters the catenary zone</td>
</tr>
<tr>
<td>1:31.0</td>
<td>4.0</td>
<td>7.0</td>
<td>C</td>
<td>N</td>
<td></td>
<td>ROV stops to record riser motion in the catenary zone</td>
</tr>
</tbody>
</table>

Figure 3.13 – Sketch of Marlin Trench
Figure 3.14 – Marlin Riser, Pictures A – H
3.3.3 Petrobras P-18, Marlim Field

The Petrobras P18 development is located in the Marlim field in the Campos Basin, Offshore Brazil as shown in Figure 3.15. The water depth is 900m. The SCR has an outer diameter of 10” and is used for gas export. At the time of the video the riser had been in service for two years. The video was taken during routine survey work, however the quality of the video is poor due to low visibility and camera flare.

![Figure 3.15 – Location of P-18, Marlim Field, Brazil](image)

A description of the SCR trench video is given below with a summary of the trench width, depth and features in Table 3.7. Photographs taken from the trench video are given in Figure 3.17 with a summary sketch of the SCR trench showing the approximate locations of the photographs in Figure 3.16.

The video starts with the ROV above the riser in the surface zone, with the riser embedded by 0.3D into the seabed as shown in Figure 3.17, picture A. Over the next 11s VET as the ROV flies towards the vessel a trench develops, which at 8s VET is 1D wide and 1D deep and at 11s VET is 3D wide and 1.5D deep. The trench walls are vertical near the trench lips and at 0.5D depth slope inwards towards the riser that is in the middle of the trench. At 11s VET a region of backfill and debris, lumps of...
which appear to be coral, can be seen covering the riser for a length of approximately 30D. The video shows this area in detail over the next 25s and is shown in picture B. In this area, when the ROV camera zooms out to view most of the trench, remnants of a second, larger trench that is 6D wide and 2D deep can be seen that surrounds the current trench. After this the ROV continues its journey towards the vessel and the trench is observed to be 3D wide, 1.5D deep with sides sloped at approximately 45° as shown in picture C. Further along the riser the larger trench observed previously begins to joint with the current trench to form a single trench near the trench mouth as shown in picture D. The video then stops with the riser near the TDP.

The next section of video for the P18 riser starts in the catenary zone and follows the riser towards the TDP, but does not enter the buried zone. The video shows that the trench mouth is 16D wide and 2D deep as shown in pictures E and F. The ROV lands on the seabed next to the left hand side trench lips, then lifts off and over to the right hand side where the riser can be seen against the trench wall moving side-to-side by 0.5D. The floor of the trench mouth has a number of shallow ridges that can be seen in picture F.
<table>
<thead>
<tr>
<th>VET (min:s)</th>
<th>Trench Depth (D)</th>
<th>Trench Width (D)</th>
<th>Picture</th>
<th>Zone</th>
<th>Backfill</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>-</td>
<td>-</td>
<td>S</td>
<td>N</td>
<td>-</td>
<td>Riser sits on the seabed at a shallow embedment depth</td>
</tr>
<tr>
<td>2.0</td>
<td>-</td>
<td>-</td>
<td>A</td>
<td>S</td>
<td>N</td>
<td>Riser sits in a shallow trench</td>
</tr>
<tr>
<td>8.0</td>
<td>1.0</td>
<td>1.0</td>
<td>B</td>
<td>N</td>
<td>-</td>
<td>Riser covered by backfill. ROV hovers around this region which is 30D long. A larger trench is outside of the current riser trench.</td>
</tr>
<tr>
<td>11.0</td>
<td>1.5</td>
<td>3.0</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>19.5</td>
<td>2.0</td>
<td>3.0</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>2.0</td>
<td>3.0</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>37.5</td>
<td>1.5</td>
<td>3.0</td>
<td>C</td>
<td>B</td>
<td>N</td>
<td>45° sloped sided trench</td>
</tr>
<tr>
<td>50.0</td>
<td>1.5</td>
<td>3.0</td>
<td>B</td>
<td>N</td>
<td>-</td>
<td>The trench combines with the larger trench</td>
</tr>
<tr>
<td>1:00.2</td>
<td>3.0</td>
<td>7.0</td>
<td>D</td>
<td>B</td>
<td>N</td>
<td>End of video 1</td>
</tr>
<tr>
<td>0</td>
<td>-</td>
<td>-</td>
<td>C</td>
<td>N</td>
<td>-</td>
<td>Start of video 2, riser in catenary zone</td>
</tr>
<tr>
<td>22.4</td>
<td>4.0</td>
<td>16.0</td>
<td>E</td>
<td>C</td>
<td>N</td>
<td>2 halves of the same trench</td>
</tr>
<tr>
<td>38.4</td>
<td>4.0</td>
<td>16.0</td>
<td>F</td>
<td>C</td>
<td>N</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 3.16 – Sketch of Petrobras P-18 Trench**
Figure 3.17 – P18 Riser Trench, Pictures A - F

3.3.4 Auger

The Auger field is located in the Gulf of Mexico, 255 miles south east of Houston and 214 miles south west of New Orleans as shown in Figure 3.18. The field is in 872m water depth and the vessel is a catenary moored tension leg platform (TLP). The export SCR has an outer diameter of 12 ¾”. The trench survey was conducted as part of routine survey work.
The Auger video shows SCR trenches from two SCRs attached to the Auger platform. A summary of the observations from the video is given in Table 3.8 with photographs taken from the video for the first and second trenches given in Figure 3.20 and Figure 3.21 respectively. A summary of the approximate locations of the photographs is given in Figure 3.19.

The first SCR trench in the video starts with the ROV in the surface zone pointing towards the vessel. The riser is observed to be sitting on top of the soil with very little embedment occurring as shown in Figure 3.20, picture A. As the ROV flies along the riser towards the vessel a sloped sided trench begins to form that at 17.7s VET is 2.5D wide and 2.0D deep as shown in picture B. As the ROV continues along the riser the trench deepens to 3.5D, the width remains constant, and the trench walls get steeper. At 27.0s VET the trench walls are vertical for a height of 1.5D near the trench floor, then sloped at 55° to the horizontal. Additionally, the riser is observed to have eroded the bottom of the right hand side trench wall so that the top of the trench overhangs the pipe by 0.1D. This is shown in picture C. As the ROV flies towards the vessel the trench walls become sloped and the trench deepens to 4.0D and widens to 5.0D at 1:07.0 VET as shown in picture D. This trench shape continues until the end of this fly-by at 1:36 VET.
The second trench survey starts in the buried zone with the riser embedded in the bottom of the trench to a depth of 0.5D as shown in Figure 3.21, picture E. The trench is 4D deep and 5D wide and bowl shaped (the trench walls are steep near the trench lips and become more gradually sloped with increasing depth). As the ROV flies towards the vessel the left hand trench wall becomes near vertical while the right hand trench wall remains bowl shaped, and the trench width reduces to 4D. The soil on the left hand side of the riser increases in height so that at 1:52.0 VET the soil covers the left side of the riser as shown in picture F. The ROV pans to the left of the trench and a second, parallel trench is observed that is 4D away and is shown in picture F. As the ROV continues along the trench the riser starts to cut into and erode the left hand trench wall making the top of the trench wall overhang the riser. The distance between the two trenches is reduced as shown in picture G. The trench also reduces in depth as the riser erodes into the left hand trench wall. At 2:17.4 VET the left hand trench wall has been eroded to a width of 2D and resembles a dividing wall within a larger trench instead of undisturbed seabed. This is shown in picture H where the riser can also be seen to be in the catenary zone. As the riser lifts away from the seabed the wall between the two trenches is sloped, and the two trenches combine into a wide and shallow trench mouth with a rippled and uneven trench floor.
Table 3.8 – Observations of Auger Trench

<table>
<thead>
<tr>
<th>VET (min:s)</th>
<th>Trench Depth (D)</th>
<th>Trench Width (D)</th>
<th>Picture</th>
<th>Zone</th>
<th>Backfill</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Surface zone</td>
</tr>
<tr>
<td>9.7</td>
<td>1.0</td>
<td>1.5</td>
<td>A</td>
<td>B</td>
<td>N</td>
<td>ROV hovers around a section of the riser</td>
</tr>
<tr>
<td>14.0</td>
<td>1.5</td>
<td>2.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17.7</td>
<td>2.0</td>
<td>2.5</td>
<td>B</td>
<td>B</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>27.0</td>
<td>3.0</td>
<td>2.5</td>
<td>B</td>
<td>N</td>
<td></td>
<td></td>
</tr>
<tr>
<td>47.6</td>
<td>3.5</td>
<td>2.5</td>
<td>C</td>
<td>B</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>53.9</td>
<td>3.5</td>
<td>2.5</td>
<td>B</td>
<td>N</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:07.0</td>
<td>4.0</td>
<td>5.0</td>
<td>D</td>
<td>B</td>
<td>N</td>
<td>Trench walls slopped at 50°</td>
</tr>
<tr>
<td>1:30.0</td>
<td>3.0</td>
<td>3.0</td>
<td>B</td>
<td>N</td>
<td></td>
<td>End of first trench fly-by</td>
</tr>
<tr>
<td>1:36.0</td>
<td>4.0</td>
<td>5.0</td>
<td>E</td>
<td>B</td>
<td>I</td>
<td>Second trench starts</td>
</tr>
<tr>
<td>1:52.0</td>
<td>4.0</td>
<td>4.0</td>
<td>F</td>
<td>B</td>
<td>I</td>
<td></td>
</tr>
<tr>
<td>2:12.0</td>
<td>2.0</td>
<td>2.5</td>
<td>G</td>
<td>B</td>
<td>I</td>
<td>Close fitting inner trench within larger trench</td>
</tr>
<tr>
<td>2:17.4</td>
<td>1.2</td>
<td>2.0</td>
<td>H</td>
<td>C</td>
<td>N</td>
<td>Distance between trenches 1.5D</td>
</tr>
<tr>
<td>2:26.0</td>
<td>1.2</td>
<td>2.0</td>
<td>I</td>
<td>C</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>2:36.4</td>
<td>1.2</td>
<td>2.0</td>
<td>C</td>
<td>N</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.19 – Sketch of Auger Trench
Figure 3.20 – First Auger Trench, Pictures A - D
3.4 Discussion

3.4.1 Introduction

The discussion of the observations from the ROV trench surveys is divided into sections. The first section presents the general shape of the SCR trenches with the ranges of widths and depths observed, which is a generic SCR trench profile and can be modelled in SCR analysis. The second section details the specific trench features, such as backfill and tension cracks that are observed and provides conclusions on
probable trenching mechanisms. The last section assesses the effects of riser/trench interaction using the trench profile in SCR design.

### 3.4.2 Generic SCR Trench Profile

The SCR trench ROV survey videos show that SCR trenches have the same general shape that is best described as being ladle shaped when viewed in profile and bell mouth shaped when viewed in plan. This shape can be seen in all of the trench surveys. A summary of the riser zone shown in the trench photographs is given in Table 3.9 and indicates where the information for the general SCR trench shape is derived. A sketch of the general trench shape is given in Figure 3.22 and described below.

- SCR trench profile – ladle shaped. The trench depth tends to be deepest near the trench mouth between the catenary zone and the buried zone, around the TDP. The trench depth reduces further along the riser towards the flowline, where in the surface zone the trench depth is zero, and the pipe is embedded in the seabed.

- SCR trench plan – bell mouth shaped, being widest near the trench mouth in the catenary zone. The trench width then tapers in along the riser towards the flowline. In the surface zone the trench width is the bearing width.

![Figure 3.22 – Overview of Trench Definitions](image)
Typical cross-sections of the trench mouth, the buried zone and near the surface zone are given in Figure 3.23, Figure 3.24, Figure 3.25 and Figure 3.26. These show the range of dimensions observed in the ROV surveys. The maximum trench dimensions from the ROV surveys are summarised in Table 3.11. A summary of the trench photographs grouped by steep or sloped wall is shown in Table 3.10. This shows that for most of the trenches the trench walls are vertical near the TDP and become more sloped the further the distance from the TDP. The exceptions are in relatively young trenches such as the Allegheny seven month’s trench where the trench walls are always steep, where there is a trench within a trench as in the Auger trench, or where the riser has begun to erode or dig into a trench wall as in the Allegheny production riser Trench B.

Table 3.9 – Zone of the Riser in Trench Photographs

<table>
<thead>
<tr>
<th>Zone</th>
<th>Allegheny Oil Export</th>
<th>Allegheny Gas Export</th>
<th>Allegheny Production</th>
<th>Marlin</th>
<th>P-18, Marlim</th>
<th>Auger</th>
</tr>
</thead>
<tbody>
<tr>
<td>Catenary Zone</td>
<td>A</td>
<td>A</td>
<td>E, F</td>
<td>E, F</td>
<td>D</td>
<td>E, F</td>
</tr>
<tr>
<td>Surface Zone</td>
<td>F</td>
<td>E</td>
<td></td>
<td></td>
<td>A</td>
<td></td>
</tr>
</tbody>
</table>
Figure 3.23 – Shape of the End of the Trench Near the Trench Mouth

Figure 3.24 – General Shape of Trench Mouth

Figure 3.25 – General Shape of Trench in Buried Zone

Figure 3.26 – General Shape of Trench Near Surface Zone
Table 3.10 – Summary of Shape of Trench Wall Sides

<table>
<thead>
<tr>
<th></th>
<th>Allegheny Oil Export</th>
<th>Allegheny Gas Export</th>
<th>Allegheny Production</th>
<th>Marlin</th>
<th>P-18, Marlim</th>
<th>Auger</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sloped Wall</td>
<td>-</td>
<td>D</td>
<td>A, B, C, D, F</td>
<td>A</td>
<td>A, C</td>
<td>B, C, D, E, F, G, H</td>
</tr>
</tbody>
</table>

Catenary zone indicated in red, locations in the buried zone and near the catenary zone in bold.

Table 3.11 – Summary of Widths and Depths from Riser Trenches

<table>
<thead>
<tr>
<th></th>
<th>Allegheny Oil Export</th>
<th>Allegheny Gas Export</th>
<th>Allegheny Production</th>
<th>Marlin</th>
<th>P-18, Marlim</th>
<th>Auger</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trench Mouth Width</td>
<td>10D</td>
<td>5D</td>
<td>15D</td>
<td>6D</td>
<td>10D</td>
<td>16D</td>
</tr>
<tr>
<td>Trench Mouth Depth</td>
<td>2D</td>
<td>1.6D</td>
<td>1D</td>
<td>1.5D</td>
<td>4D</td>
<td>4D</td>
</tr>
<tr>
<td>Maximum Depth</td>
<td>4.5D</td>
<td>3D</td>
<td>5D</td>
<td>4D</td>
<td>4D</td>
<td>3D</td>
</tr>
<tr>
<td>Width in Buried Zone</td>
<td>4D</td>
<td>3.5D</td>
<td>4.5D</td>
<td>5D</td>
<td>5D</td>
<td>3D</td>
</tr>
</tbody>
</table>

3.4.3 Trench Features

A number of SCR trench features, such as trench in trench and backfill, are observed in the ROV trench surveys that include tension cracks in the seabed, soil overhanging the riser and backfill. A summary of these features with the trench survey picture where they are observed in given in Table 3.12 and detailed below.

Backfill is the sediment that covers the riser in the trench. Backfill can be created by the day-to-day motions of the riser eroding the trench, by trench walls collapsing on top of the riser, which is described below, or by sediment washed into the trench and deposited on top of the riser by seabed currents.
Table 3.12 – Summary of Observations from ROV Trench Surveys

<table>
<thead>
<tr>
<th>Smooth Sides</th>
<th>Allegheny Oil Export</th>
<th>Allegheny Gas Export</th>
<th>Allegheny Production</th>
<th>Marlin</th>
<th>P-18, Marlim</th>
<th>Auger</th>
</tr>
</thead>
<tbody>
<tr>
<td>Close Fitting Trench</td>
<td>-</td>
<td>D</td>
<td>-</td>
<td>B</td>
<td>F</td>
<td>C</td>
</tr>
<tr>
<td>Backfill</td>
<td>B, C, D, E</td>
<td>-</td>
<td>D, E</td>
<td>-</td>
<td>B</td>
<td>F</td>
</tr>
<tr>
<td>Trench in Trench</td>
<td>-</td>
<td>-</td>
<td>B, E, F</td>
<td>A, B, C</td>
<td>A, H</td>
<td>-</td>
</tr>
<tr>
<td>Tension Cracks</td>
<td>C, D</td>
<td>C, D</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Overhang</td>
<td>-</td>
<td>-</td>
<td>B, C</td>
<td>F</td>
<td>-</td>
<td>C, H</td>
</tr>
</tbody>
</table>

Close fitting trenches are formed near the surface zone of the TDZ where there is little riser motion. They are also observed in existing trenches where the riser has penetrated by settlement or erosion by the vibration from small day-to-day riser TDP motions into the trench floor.

The trench in trench feature occurs when a smaller trench forms inside an existing larger one as shown in Figure 3.27. Mechanisms that could cause the trench in trench feature are that the riser has experienced large motions in its past, or during installation, that have widened the trench. The day-to-day motions of the riser are typically small and the erosion of the trench is then focused in a smaller area, creating a trench within a trench.

Figure 3.27 – Trench in Trench
Tension cracks on the seabed surface are observed in both of the Allegheny export riser trenches. The trench surveys show that the tension cracks occur when the trench walls are steep and the trenches are relatively young, less than two years old. It is also observed that the trench walls have collapsed in line with the tension cracks, covered the riser with backfill and widened the trench. This is illustrated in Figure 3.28 and indicates a probable trenching mechanism that could be predicted using slope stability mechanics.

Another mechanism for backfill and trench enlargement is for the SCR to erode and/or penetrate into the trench wall and create an overhang as observed in the Allegheny production riser, Marlin and Auger riser trenches. A sketch of this is shown in Figure 3.28. After a time the overhang collapses, covers the riser with backfill and the trench widens.

![Figure 3.28 – Assumed Mechanisms for Backfill](image)

3.4.4 Development of Analytical Trench Model

Existing trench models used in SCR analysis (Thethi & Moros, 2001) assume a consistent trench profile along the riser length that is about 3D deep, between 1D to 3D wide with vertical sides as shown in Figure 3.29. This conservatively models the trench so that any riser/trench interaction will have the maximum effect on riser stress. Further description of the methods used to model SCR trenches are given in the previous section.
Using the TDP location study presented by Thethi & Moros (2001), which is summarised in Table 3.13, the conservative trench model indicates that all riser motions cause the riser to interact with the trench wall. The evidence from the trench surveys shows that this steep sided type of trench does exist in young trenches, such as Allegheny. However the width of the trench tends to be greater than 3.5D at the TDP, as shown in Figure 3.25. A more typical example of a steep sided trench occurs with a trench in trench or overhanging trench profile, shown in Figure 3.27 and Figure 3.28 respectively, and then only one of the trench walls is vertical. This implies that modelling only one steep sided trench wall, as shown in Figure 3.30, creates a less conservative trench model.

**Table 3.13 – Summary of Distance and Occurrence of SCR TDP Motions, Thethi & Moros (2001)**

<table>
<thead>
<tr>
<th>Motion</th>
<th>Probability of Occurrence</th>
<th>Limit of In-Plane TDP Motions (Near-Far Axis)</th>
<th>Limit of Transverse TDP Motions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Day-to-day</td>
<td>95%</td>
<td>±43D</td>
<td>±0.5D</td>
</tr>
<tr>
<td>Extreme Storm</td>
<td>2.5%</td>
<td>±70D</td>
<td>±1D</td>
</tr>
<tr>
<td>Second Order Vessel Motions</td>
<td>2.5%</td>
<td>±200 – 260D</td>
<td>±7D</td>
</tr>
</tbody>
</table>
3.4.5 Influence of SCR Trench on Riser Analysis

The aims, among others, of the trench surveys is to obtain better SCR trench profiles for use in SCR analysis to more accurately calculate riser stresses and fatigue lives. Consequently as assessment is required to determine the significance of SCR trenches on the SCR during day-to-day, extreme storm and second order motions. The assessment is conducted using the SCR dynamic TDP envelope given by Thethi & Moros (2001) and assumes that the majority of the TDP motion will be near the trench mouth. The conclusions are summarised below:

- Day-to-day motions (which occur for 95% of the risers life) have low in-plane and transverse TDP motions that may not interact with the trench wall. This indicates that for the majority of SCR analysis the trench can be ignored.

- Vessel motions predominantly in the riser plane will not cause the SCR to interact with the trench wall. Consequently for this type of motion the trench can be considered to be a sloping flat seabed.

- Small vessel motions, such as day-to-day motions due to small storm waves, in the transverse axis may not cause large motions at the TDP and consequently may not cause riser/trench interaction. It is assumed that if pipe/trench interaction did occur from this type of motion there would be a small increase in bending stress in the transverse axis. This has little or no effect on the in-plane axis that has the large TDP bending stress. Additionally this type of motion would erode the trench wall at the point of riser/trench interaction and reduce the out-of-plane bending stress.

- For extreme storm wave where transverse TDP motions could be large it is possible that riser/trench interaction may occur. However, as for the small vessel motion, the bending is in the out-of-plane axis and the effect on SCR design is assumed to be negligible.

- Large second order vessel motions in the transverse axis would cause SCR riser/trench interaction. The SCR would bend around the trench wall, causing
a local increase in bending moment and hence riser stress. It is thought that this type of riser motion requires a trench model.

These observations indicate that for the majority of SCR motions the trench profile is not required in SCR analysis. However if the motions are predominantly in the transverse direction and are large, such as slow drift and second order vessel motions then the trench profile should be considered. This is summarised in Table 3.14.

<table>
<thead>
<tr>
<th>Motion</th>
<th>Probability of Occurrence</th>
<th>Riser Motions in Near-Far Axis</th>
<th>Riser Motions in Transverse Axis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Day-to-day</td>
<td>97%</td>
<td>Small</td>
<td>Small</td>
</tr>
<tr>
<td>Extreme Storm</td>
<td>2.7%</td>
<td>Small</td>
<td>Small to Medium</td>
</tr>
<tr>
<td>Second Order</td>
<td>0.3%</td>
<td>Small</td>
<td>Large</td>
</tr>
</tbody>
</table>

3.5 Summary and Conclusions

The SCR trench surveys provide a valuable insight into SCR trench geometry and evidence of the mechanisms that create SCR trenches. These can be used to better predict the trench shape, and therefore the stress distribution along the TDZ that could improve the design of SCRs.

SCR trenches are observed to be ladle shaped in profile and bell mouth shaped in plan, they are widest near the catenary zone, then taper towards the surface zone where the riser acts as a static pipeline. The trench depths are observed to be up to six diameters deep near the TDP, and then taper to zero diameters depth in the surface zone. The trenches are observed to be both steep sided and sloped, with steep sided trench walls generally occurring near the trench mouth and the catenary side of the buried zone. Further away form the trench mouth the trench walls tend to be sloped.
Observations from the trench surveys have provided evidence of backfill, tension cracks, overhang, close fitting trenches and the trench in trench feature.

The existing analytical trench model is updated based on the observations from the trench surveys to reduce its conservatism. The significance of riser/trench interaction is assessed and it is recommended that SCR trenches should be considered when the SCR is subject to large transverse motions such as slow drift.

The exact trenching mechanisms can only be inferred. To completely understand subsea trenching mechanisms regular surveys of SCR trenches is required to observe the effects of riser motion and subsea currents on the trench shape. This type of study can be conducted on installed risers or using a full scale test riser on a subsea mud where the trench can be examined regularly.
4.0 HARBOUR TEST RISER

4.1 Introduction

As part of the STRIDE JIP (Steel Risers in Deepwater Environments Joint Industry Project) which was conducted by 2H Offshore Ltd (2001b) a series of full scale tests were conducted to investigate the effects of seabed interaction on catenary riser response and wall stress. The tests were conducted in Watchet Harbour, Somerset, UK as the harbour geotechnical properties, such as undrained soil shear strength and plasticity index, reported by Fugro Limited (1998) are similar to deepwater Gulf of Mexico (GoM) sediments.

The full-scale harbour test riser arrangement consisted of a 110m long, 0.1683m (6-5/8") outer diameter with 6.9mm wall thickness steel pipe, supported from a harbour wall and anchored on the seabed as shown in Figure 4.1.

Figure 4.1 – Full Scale Test Riser

The harbour test riser was configured to simulate the bottom 10m of a SCR in 1000m water depth with a 12° top angle connected to a spar vessel as shown in Figure 4.2. At the harbour wall the riser is actuated to replicate the touchdown point (TDP) motions. The harbour is tidal, ranging from empty to 5m water depth, and this ability to test with and without water provided an opportunity to evaluate different aspects
of TDP interaction both in isolation and combination. Strain gauges were used to measure the bending moments at 13 positions near the TDP and a load cell was used to measure the tension in the riser near the actuator.

Figure 4.2 – SCR Section Modelled by Harbour Test Riser

4.2 Harbour Test Riser Set up

4.2.1 Harbour Test Riser Properties

An 110m long riser was draped from an actuator positioned on the harbour wall, across the clay seabed to an anchor as shown in Figure 4.3. The pipe properties are summarised in Table 4.1 and the in air and submerged pipe weights given in Table 4.2. The position of the anchor and the actuator were measured on site using a total station with electromagnetic distance measurement (EDM) capabilities.
To position the pipe on the undisturbed soil it was floated into position using temporary buoyancy and anchored at the bottom end to a pattern of mud anchors. The top end was then pulled in on a winch before the buoys were removed and the pipe allowed to settle as the tide went out. Connection was then slowly transferred from the winch to the actuator. The starting point for the riser configuration depended on getting the tension calculated by the pre-analysis which is detailed in 2H Offshore (2001b). By using different chain links in the connection rigging and a 15 tonne turn-buckle the pipe configuration could be tuned to obtain the final level of pipe tension, and get the nominal TDP at the required position.

![Figure 4.3 – Harbour Test Riser](image)

**Table 4.1 – Summary of Test Rig Parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe outer diameter</td>
<td>0.1683m (6-5/8&quot;)</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>6.9mm</td>
</tr>
<tr>
<td>Pipe material</td>
<td>APL 5L Grade B, 448.2x10^6 N/m² yield</td>
</tr>
<tr>
<td>Height of nominal position above seabed</td>
<td>9.65m (10.29m Above MSL)</td>
</tr>
<tr>
<td>Length of chain at actuator</td>
<td>3.85m</td>
</tr>
<tr>
<td>Length of pipe</td>
<td>113m</td>
</tr>
<tr>
<td>Mean high tide water level</td>
<td>3.5m</td>
</tr>
</tbody>
</table>
Table 4.2 – Summary of Mass per Unit Length for All Riser Configurations

<table>
<thead>
<tr>
<th>External Fluid</th>
<th>Internal Fluid</th>
<th>Mass per unit length (kg/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air</td>
<td>Air</td>
<td>27.5</td>
</tr>
<tr>
<td>Air</td>
<td>Water</td>
<td>46.7</td>
</tr>
<tr>
<td>Sea Water</td>
<td>Air</td>
<td>4.7</td>
</tr>
<tr>
<td>Sea Water</td>
<td>Water</td>
<td>23.9</td>
</tr>
</tbody>
</table>

The actuator comprised of a heavy duty truss frame with a 30 kVA motor turning a 3m long ball screw, Figure 4.4. The riser pipe was connected to a ball screw nut that was then driven backwards and forwards as the screw turned. The motor was controlled by a custom designed PLC drive system with full feedback control to provide the prescribed linear and sinusoidal motions at the top of the pipe. The system had to be capable of full dynamic braking in the event of power loss, and able to dissipate as well as generate the considerable power associated with each actuation cycle.

Figure 4.4 – Actuator
The rail supporting the ball screw could be unbolted to pivot around its centre to provide pipe actuation in the vertical or horizontal planes. In addition the entire actuator truss frame sat on wheels that allowed it to be moved sideways to access different test corridors and to simulate lateral vessel drift. The riser tension was typically 8 to 12 tonnes and all equipment had to be designed to cope with this in static and dynamic modes.

4.2.2 Instrumentation

Strain gauges were positioned along the pipe as indicated in Figure 4.5 (2H Offshore, 2000a) and Figure 4.6. Strain gauge A was 46.41m from the top end of the pipe (50.41m from the plane of actuation). All gauges were configured as full bridges to provide the bending strain at particular pipe sections, i.e. they were positioned on diametrically opposite walls of the pipe, and were independent of local pipe axial tension. Some of the positions measured the in-plane bending only, others had out-of-plane also. The only reliable gauges were those shown in red, which are gauges A, C, D, F, J, K and M. The strain gauges at locations B, E, G, H, I and L suffered from water ingress.

Axial tension was recorded using load cells at the top and bottom ends of the pipe. Vertical and horizontal bending strain were also recorded by two strain gauge bridges attached to a travelling encastre type support at the connection between the top of the pipe and actuator saddle, which was known as the ‘bending beam’. A triaxial accelerometer unit was mounted on the pipe above the TDP, 65m from the anchor (bottom) end of the pipe. The actuator position was monitored and recorded by an ultrasonic distance measurement device mounted on the actuator, and sounding off a reference plate. All instrumentation was hardwired to an Instrunet (GWI, 1998) interface and signal conditioning system connected to a PC running DASYLab (DASYTEC, 1996) and logging at a frequency of 10Hz.

The position of the actuator was measured after each test using a total station. At low tides it was also possible to measure the pipe position and the trench deformation.
4.2.3 Marine and Geotechnical Properties

The mean sea level was 3.5m above the anchor and the harbour was free flooding. The current velocity due to the tides in the test area as the harbour filled and emptied was low. Tests were conducted at both high and low tides. This allowed for TDP to
be observed during the low tide tests and for detailed geotechnical and trench surveys to be conducted.

Watchet Harbour was chosen for the STRIDE III full scale tests because the clay soil within the harbour was similar to the clay found in the Gulf of Mexico (GoM). The Watchet Harbour clay is described as a very soft dark grey-to-grey clay, ranging in thickness from zero to 2.0m thick, Fugro (1998). This description and the further laboratory tests confirmed that the Watchet Harbour clay was consistent with the high plasticity marine clays found in deepwater GoM.

During the full scale experiments more detailed tests were conducted along the test corridors. These included bore hole samples which were taken away for detailed laboratory analysis and in-situ tests using a shear vane the writer designed (2H Offshore, 2000c) specifically for low strength clays (<20kPa). A summary of the Watchet Harbour geotechnical parameters obtained with typical values for Gulf of Mexico clays from Fugro (1999) and 2H Offshore (2000c) are given in Table 4.3. Detailed soil shear strength measurements were obtained using a shear vane as shown in Figure 4.7. Soil shear strength is considered to be a key parameter that influences the TDP behaviour. A thorough seabed survey using a total station was also carried out and the resulting seabed profile along the harbour test riser length is given in Figure 4.8.
Table 4.3 – Soil Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Watchet Harbour Clay</th>
<th>Gulf of Mexico Clay, Fugro (1999)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content, w</td>
<td>104.7%</td>
<td>-</td>
</tr>
<tr>
<td>Bulk Density, ( \rho )</td>
<td>1.46 Mg/m(^3)</td>
<td>-</td>
</tr>
<tr>
<td>Dry Density, ( \rho_d )</td>
<td>0.73 Mg/m(^3)</td>
<td>-</td>
</tr>
<tr>
<td>Particle Density, ( \rho_s )</td>
<td>2.68 Mg/m(^3)</td>
<td>-</td>
</tr>
<tr>
<td>Liquid Limit, ( w_L )</td>
<td>87.6%</td>
<td>-</td>
</tr>
<tr>
<td>Plastic Limit, ( w_P )</td>
<td>38.8%</td>
<td>-</td>
</tr>
<tr>
<td>Plasticity Index, ( I_p )</td>
<td>48.9%</td>
<td>50%</td>
</tr>
<tr>
<td>Average Organic Content</td>
<td>3.2%</td>
<td>-</td>
</tr>
<tr>
<td>Specific Gravity, ( G_s )</td>
<td>2.68</td>
<td>-</td>
</tr>
<tr>
<td>Coefficient of Consolidation, ( c_V ) at ( \frac{1}{4}D )</td>
<td>0.5 m(^2)/year</td>
<td>-</td>
</tr>
<tr>
<td>Coefficient of Volume Compressibility, ( m_V ) at ( \frac{1}{2}D )</td>
<td>15 m(^2)/MN</td>
<td>-</td>
</tr>
<tr>
<td>Undisturbed Shear Strength at ( \frac{1}{2}D )</td>
<td>2.86 kPa</td>
<td>1.3 – 4.3</td>
</tr>
<tr>
<td>Remoulded Shear Strength at ( \frac{1}{2}D )</td>
<td>0.87 kPa</td>
<td>0.43 – 1.43</td>
</tr>
<tr>
<td>Sensitivity of Clay at ( \frac{1}{2}D )</td>
<td>3.3</td>
<td>3.0</td>
</tr>
</tbody>
</table>

**Notes:**
Riser diameter, D, is taken as 0.1683m
Figure 4.7 – The Writer Conducting Shear Vane Tests in the Soil on the Harbour Test Riser

Figure 4.8 – Seabed Profile Measured Using a Total Station
4.2.4 Test Corridors

The full scale tests were conducted to examine the effects of 3D pipe/soil interaction, in particular pipe/soil suction and riser/trench interaction. The experiments were conducted in a number of test corridors that had different trench conditions, including an open trench formed by the presence of the harbour test riser, an artificially deepened trench, a backfilled trench and a rigid seabed, Table 4.4. Photographs of the harbour test riser in the test corridors are given in Figure 4.9 to Figure 4.12. The test corridors were close together and the geotechnical properties were found to be identical. This allowed for direct comparison between the results, indicating that any differences would be due to the different riser trenches. A rigid seabed was created using Perforated Steel Planks (PSP) placed over an existing trench. The rigid seabed also removed the influence of the soil away from the riser and served as a benchmark for finite element analysis (FEA) comparisons.

<table>
<thead>
<tr>
<th>Test Corridors</th>
<th>Test Corridor Title</th>
<th>Description/Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>First Trench</td>
<td>Initial trials, no data recorded</td>
</tr>
<tr>
<td>2</td>
<td>Open Trench</td>
<td>Formed naturally by riser self weight and vertical/lateral riser motions</td>
</tr>
<tr>
<td>3</td>
<td>Artificially Deepened Trench</td>
<td>The open trench was artificially deepened by hand to approximately 3D</td>
</tr>
<tr>
<td>4</td>
<td>Backfilled Trench</td>
<td>The artificially deep trench was backfilled with clay, covering the riser</td>
</tr>
<tr>
<td>5</td>
<td>Rigid Seabed</td>
<td>Perforated Steel Planks (PSP) were placed over the trench and under the riser to simulate a rigid seabed</td>
</tr>
</tbody>
</table>
Figure 4.9 – Harbour Test Riser in Open Trench Formed by Riser Motion, Looking from Anchor to Actuator

Figure 4.10 – Artificially Deepened Trench Showing the Curvature of the Harbour Test Riser
Figure 4.11 – Harbour Test Riser in Backfilled Trench

Figure 4.12 – Harbour Test Riser on PSP for Rigid Seabed Tests
4.2.5 Test Description

The top of the harbour test riser was moved vertically and horizontally to represent vessel motions during slow drift, day-to-day wave and extreme wave vessel motions. Analysis conducted on the SCR, by the writer and detailed in 2H Offshore Engineering Ltd (2001b), determined the approximate riser motions at 10m above the seabed. These motions were then simplified so that the actuator could represent them. The slow drift motions consisted of the following actuator motions, details of which are given in Table 4.5 and illustrated in Figure 4.13.

- Vertical pull up – where the actuator lifts the riser from the bottom of the frame to the top of the frame
- Lay down – where the actuator lowers the riser from the top of the frame to the bottom of the frame
- Lateral pull-out – where the actuator pulls the top of the riser laterally using the complete range of the frame

The wave induced motions were vertical or lateral sine waves at the different ‘dynamic’ offset positions to allow for actuator stroke. The dynamic actuator motions (amplitudes and periods) are given in Table 4.6. A plot of the recorded vertical dynamic actuator motions for the near, nominal and far extreme storm motions is given in Figure 4.14. This shows that the actuator system lead the riser through a series of regular sine waves, overcoming the inertia forces from the riser.

![Figure 4.13 – Effect of Actuator Near and Far Offsets on Riser Configuration](image)
### Table 4.5 – Actuator Positions with Equivalent SCR Offsets

<table>
<thead>
<tr>
<th>Actuator Position</th>
<th>Vertical Actuator Stroke from Nominal (m)</th>
<th>Equivalent Vessel Offset Position (% Water Depth)</th>
<th>TDP Movement from Nominal (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near</td>
<td>-0.8</td>
<td>-0.8</td>
<td>-11.0</td>
</tr>
<tr>
<td>Near – Dynamic</td>
<td>-0.5</td>
<td>-0.5</td>
<td>-6.3</td>
</tr>
<tr>
<td>Nominal</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Far – Dynamic</td>
<td>1.0</td>
<td>1.0</td>
<td>13.7</td>
</tr>
<tr>
<td>Far</td>
<td>1.4</td>
<td>1.4</td>
<td>20.6</td>
</tr>
</tbody>
</table>

**Note:**
Equivalent Vessel Offset Position is the equivalent vessel offset corresponding to the actuator position.

### Table 4.6 – Actuator Motion with Equivalent Simulated Vessel Motions

<table>
<thead>
<tr>
<th>Motion</th>
<th>Actuator Amplitude (m)</th>
<th>Wave Amplitude (m)</th>
<th>Period (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Day to Day</td>
<td>±0.05</td>
<td>±1.2</td>
<td>6</td>
</tr>
<tr>
<td>Extreme</td>
<td>±0.4</td>
<td>±8.0</td>
<td>25</td>
</tr>
<tr>
<td>Lateral Extreme</td>
<td>±0.5</td>
<td>±8.0</td>
<td>18</td>
</tr>
</tbody>
</table>
**4.2.6 Test Numbering**

Each test was labelled using a unique identifier that consisted of three sections, A, B and C in the form A-BC. A was a numeric from 1 to 5 which identified the test corridor number; B was a number from 1 to 20 which identified the test series and C was a blank or a letter which identified where the test occurred in a sequence (a blank indicated the first test, A the second, B the third, etc). For example the test label 4-1 indicates the test was conducted in test corridor 4 (backfilled trench) and was the first test conducted in that trench in that series, whereas test label 3-5C indicates that the test was conducted in test corridor 3 (artificially deepened trench), was the 5th test conducted in that series and was the 4th test in that sequence.

**4.2.7 Test Program**

The test program is summarised for the in water tests and the in air tests in Table 4.7 and Table 4.8 respectively. These show that pull up and lay down tests were conducted on every test corridor, however, due to time limitations the wave motions were only conducted on test corridor 2 (open trench) and test corridor 5 (rigid seabed). The pull up and associated lay down tests were typically conducted as a series of five consecutive pairs. The first pull up test is considered to be on
undisturbed clay as the riser was allowed to consolidate the clay soil in the trench. The term consolidation time refers to the length of time that the riser was in contact with the seabed prior to the pull up test. The consolidation time and the sea level of the first pull up tests are shown in Table 4.9. The subsequent tests in the pull up and lay down series are considered to be on remoulded clay. All pull up tests were conducted with a pull up speed of 0.1m/s, except test 2-1E which had a pull up speed of 0.01m/s.

Table 4.7 – Matrix of Tests In Water Tests with Reference Numbers

<table>
<thead>
<tr>
<th>Test Corridor</th>
<th>2 Open Trench</th>
<th>3 Artificial Trench</th>
<th>4 Backfilled Trench</th>
<th>5 Rigid Seabed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pull up / Lay down</td>
<td>3, 4, 7, 8, 10, 11, 13, 14 (C, D)</td>
<td>3, 4 5, 6</td>
<td>1, 2</td>
<td>1, 2</td>
</tr>
<tr>
<td>Dynamic @ Near</td>
<td>5</td>
<td>-</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td>Dynamic @ Nominal</td>
<td>6</td>
<td>-</td>
<td>-</td>
<td>4</td>
</tr>
<tr>
<td>Dynamic @ Far</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>5</td>
</tr>
<tr>
<td>Lateral Pull-out</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Lateral Dynamic</td>
<td>16</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Notes:
Tests in bold are pull up tests
Table 4.8 – Test Matrix of In Air Tests with Test Reference Numbers

<table>
<thead>
<tr>
<th>Test Corridor</th>
<th>2 Open Trench</th>
<th>3 Artificial Trench</th>
<th>4 Backfilled Trench</th>
<th>5 Rigid Seabed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pull up / Lay down</td>
<td>1, 2, 13, 14 (A, B)</td>
<td>1, 2</td>
<td>3, 4</td>
<td>6, 7, 10, 11</td>
</tr>
<tr>
<td>Dynamic @ Near</td>
<td>9, 12</td>
<td>-</td>
<td>-</td>
<td>8, 12</td>
</tr>
<tr>
<td>Dynamic @ Nominal</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>9, 13</td>
</tr>
<tr>
<td>Dynamic @ Far</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>14</td>
</tr>
<tr>
<td>Lateral Pull-out</td>
<td>17, 18, 19, 20</td>
<td>7, 8</td>
<td>-</td>
<td>15</td>
</tr>
<tr>
<td>Lateral Dynamic</td>
<td>15</td>
<td>-</td>
<td>-</td>
<td>16</td>
</tr>
</tbody>
</table>

Notes:
Tests in bold are pull up tests

Table 4.9 – Summary of First Pull Up Tests

<table>
<thead>
<tr>
<th>Consolidation Time (hours)</th>
<th>Sea Level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.0 – 1.5</td>
</tr>
<tr>
<td>Rigid Seabed</td>
<td>5-6, 5-10</td>
</tr>
<tr>
<td>4</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>2-1, 3-1</td>
</tr>
<tr>
<td>16</td>
<td>2-13</td>
</tr>
<tr>
<td>72</td>
<td>4-3</td>
</tr>
</tbody>
</table>

4.2.8 Data Processing

The DASYLab (DASYTEC, 1996) program output three text files that contained 1) in-plane bending strain for each strain gauge location, 2) out-of-plane bending strain at each strain gauge location and 3) the actuator displacement, top and bottom tensions and the accelerations from the two tri-axial accelerometers. The DASYLab (DASYTEC, 1996) program smoothed the signals using a low pass filter to remove
4.3 Experimental Results

Examples of the experimental data for test 4-1 (pull up test in test corridor 4, backfilled trench) and test 4-2 (lay down test in test corridor 4, backfilled trench) are given in Figure 4.15 and Figure 4.16 respectively. The figures each show seven plots, four timetraces of actuator position, in-plane bending moment, out-of-plane bending moment and tension on the left hand side of the page. On the right hand side of the page are three plots of in-plane bending moment, out-of-plane bending moment and tension verses actuator position. Further experimental results can be found in 2H Offshore (2000b) and in Appendix C.

During the pull up test, Figure 4.15, the actuator position increases from −0.8m (the bottom of the actuator stroke) to 1.4m (the top of the actuator stroke) between 62.5s and 83s. Prior to any actuator movement the in-plane and out-of-plane bending moments and top and bottom tensions remain constant. However, the out-of-plane bending moment at strain gauge location K was observed to drift prior to actuation, and for this test was ignored.

The timetraces show that during the pull up actuation, shown in Figure 4.15, the out-of-plane bending moments remain constant, the top and bottom riser tensions increase linearly, while the in-plane bending moments drop to a minimum value then increase so that after the actuation finished the bending moments remain constant. The bending moments with actuator position show that the majority of the change in bending moment and tension occurred during actuation, although the bending moment at location J continued to change after the pull up actuation had finished. The lay down test, Figure 4.16, shows similar trends, however the actuator moved in the reverse direction, from the top of the actuator stroke to the bottom and all changes in bending moment occurred within the lay down actuation.
During pull up tests it was observed that as the actuator lifted the top of the riser the TDP moved towards the anchor. This meant that the riser was not lifted vertically away from the soil, but upwards and along the seabed in a unzipping or ‘peeling’ motion.

![Graphs showing actuator position vs time, in-plane and out-of-plane bending moments vs time and position, tension vs time and position](image.png)

Figure 4.15 – Summary of Test Data from Test 4-1
Test 4-2
Lay Down Test in Test Corridor 4 (Backfilled Trench)

— Strain Gauge A — Strain Gauge C — Strain Gauge D
— Strain Gauge F — Strain Gauge J — Strain Gauge K
— Strain Gauge M

Figure 4.16 – Summary of Test Data from Test 4-2
4.4 Discussion of Experimental Results

4.4.1 Bending Moment Vertical Pull Up Tests

The vertical tests were conducted to examine the effect of pipe/soil suction on the riser. The lay down test is considered to represent the 'no pipe/soil suction' case and the pull up test represents the 'with pipe/soil suction' case so that the two bending moment responses can be compared directly. The results from the harbour test riser are presented as bending moment responses versus actuator position at strain gauge locations. An example of the bending moment data from a strain gauge during a first pull up test on test corridor 2 (natural trench) and the associated lay down test is shown in Figure 4.17. A negative bending moment corresponds to a sagging bend in the riser.

Both the pull up and lay down bending moment responses start from the −0.8m actuator position with bending moments of around −0.5 kNm. The lay down bending moment response decreases steadily to a minimum value of −5.5kNm at an actuator position of 0.5m where it levels off. In contrast, the pull up bending moment response does not change until the actuator has moved to the −0.3m actuator position. This indicates that the pipe/soil suction force is holding the riser in place. The bending moment then decreases rapidly to peak at −11 kNm at an actuator position of 0.5m, which is twice the lay down bending moment. The pull up bending moment response then increases to join the lay down bending moment response at the 1.2m actuator position.

From this example, Figure 4.17, it can be seen that the peak bending moment during a near to far pull up test is twice that of the peak bending moment seen during the associated lay down test.

The bending moment response of a pull up and lay down test pair on the rigid seabed is shown in Figure 4.18. It can be seen that the rigid seabed pull up and lay down test bending moments are virtually identical to the lay down bending moment shown in
Figure 4.17. This shows that the peak in the bending moment response during the pull up test with the riser on the clay soil is due to pipe/soil suction, and not a result of the actuation system or hysteresis/inertia effects.

\[ E_z \]
\[ c > E_0 \]
\[ 5 \]
\[ Puli Up \]
\[ a t \]
\[ c > c_0 \]
\[ -10 \]
\[ -12 \]
\[ -12 \]
\[ -10 \]
\[ -10 \]
\[ -8 \]
\[ -8 \]
\[ -6 \]
\[ -6 \]
\[ -4 \]
\[ -4 \]
\[ -2 \]
\[ -2 \]
\[ 0 \]
\[ 0 \]
\[ 0.0 \]
\[ 0.5 \]
\[ 1.0 \]
\[ 1.5 \]
\[ Actuator Displacement (m) \]
\[ 1.0 \]
\[ 1.5 \]
\[ 1.0 \]
\[ 0.5 \]
\[ 0.0 \]
\[ -0.5 \]
\[ -1.0 \]
\[ -1.0 \]
\[ -12 \]
\[ -12 \]

Figure 4.17 – Suction Peak for Fast Drift Case

Comparison of Pull Up and Lay Down on Rigid Test Corridor
Tests 5-6 and 5-7

\[ -10 \]
\[ -12 \]
\[ -12 \]
\[ -10 \]
\[ -10 \]
\[ -8 \]
\[ -8 \]
\[ -6 \]
\[ -6 \]
\[ -4 \]
\[ -4 \]
\[ -2 \]
\[ -2 \]
\[ 0 \]
\[ 0 \]
\[ 0.0 \]
\[ 0.5 \]
\[ 1.0 \]
\[ 1.5 \]
\[ Actuator Displacement (m) \]
\[ -10 \]
\[ -10 \]
\[ -12 \]
\[ -12 \]

Figure 4.18 – Comparison of Pull Up with Lay down Tests in the Rigid Test Corridor
4.4.2 Bending Moments Along the Riser

The effect of pipe/soil suction on the top tension and bending moments along the riser during a first pull up and the associated lay down test on the natural trench is shown in Figure 4.20. Similar plots have been created for the artificially deepened trench, backfilled trench and rigid seabed and are given in Appendix C. A summary of the pull up and lay down tests with the bending moments from strain gauge locations presented is given in Table 4.10. The location of the strain gauges along the riser is shown in Figure 4.13.

Table 4.10 – Summary of Selected Tests

<table>
<thead>
<tr>
<th>Test Corridors</th>
<th>Pull Up Test</th>
<th>Lay Down Test</th>
<th>Rest Time Before First Pull Up Test</th>
<th>Strain Gauge Positions Presented</th>
</tr>
</thead>
<tbody>
<tr>
<td>2, Natural Trench</td>
<td>2-10</td>
<td>2-11</td>
<td>72 hours</td>
<td>A, D, F, J, K, M</td>
</tr>
<tr>
<td>3, Artificially Deepened</td>
<td>3-5</td>
<td>3-6</td>
<td>16 hours</td>
<td>A, C, D, J, K, M</td>
</tr>
<tr>
<td>4, Back Filled</td>
<td>4-1</td>
<td>4-2</td>
<td>4 hours</td>
<td>A, C, D, J, K, M</td>
</tr>
<tr>
<td>5, Rigid Seabed</td>
<td>5-6</td>
<td>5-7</td>
<td>-</td>
<td>A, C, D, J, K, M</td>
</tr>
</tbody>
</table>

The pull up tests are shown by red lines and the lay down tests by blue lines in Figure 4.19 and Figure 4.20. The actuator pull up rate was 0.1m/s for all tests and simulated a slow drift motion.

The top tensions given in Figure 4.19 show that the top tensions linearly increased from around 60kN when the actuator was at its lowest point to between 99kN and 130kN, depending on the content of the riser and weather the riser was in water or in air, when the actuator was at the highest point. A summary of the top tensions observed is given in Table 4.11.
Table 4.11 – Summary of Top Tensions Observed During Selected Harbour Tests

<table>
<thead>
<tr>
<th>Test Corridor</th>
<th>Tension at Bottom of Actuator Stroke (kN)</th>
<th>Tension at Top of Actuator Stroke (kN)</th>
<th>Tension Difference at +0.3m Actuator Position (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pull Up</td>
<td>Lay Down</td>
<td>Pull Up</td>
</tr>
<tr>
<td>2</td>
<td>59.4</td>
<td>59.4</td>
<td>114.2</td>
</tr>
<tr>
<td>3</td>
<td>54.6</td>
<td>54.6</td>
<td>99.4</td>
</tr>
<tr>
<td>4</td>
<td>51.3</td>
<td>48.7</td>
<td>101.4</td>
</tr>
<tr>
<td>5</td>
<td>69.3</td>
<td>69.3</td>
<td>129.6</td>
</tr>
</tbody>
</table>

A general description of the pull up and lay down bending moments shown in Figure 4.20, which are typical for all tests follow:

**Strain Gauge Position A** – this location was free hanging when the riser was in the near (lowest) actuation position. As the riser was pulled up the strain gauge showed a small decrease (around 0.3kNm) in the bending moment as it was pulled up into a straighter part of the catenary.

**Strain Gauge Positions D and F** – these locations were positioned close to the nominal TDP, were in contact with the seabed in the near riser position and were free hanging when the riser was pulled up. They showed the greatest change in bending moment due to pipe/soil suction in test corridors 2, 3 and 4. In test corridor 5 (rigid seabed) the bending moments were observed to be coincident.

**Strain Gauge Positions J and K** – these locations were in contact with the seabed for much of a pull up test, only becoming free hanging when the actuator position was above 1.0m. In test corridors 2, 3 and 4 they showed that the pipe/soil suction held the riser to the seabed.

**Strain Gauge Position M** – This location was in contact with the soil at both near and far actuator positions and did not show any change in bending moment due to actuator movement.
Figure 4.19 – Comparison of Pull Up and Lay Down Tensions
Naturally Formed Trench (Test Corridor 2)

Figure 4.20 – Comparison of Pull Up and Lay Down Bending Moments
Naturally Formed Trench (Test Corridor 2)
4.4.3 Trench Types

A comparison of the pull up and lay down bending moments at strain gauge location D from the different trenches is given in Figure 4.21. This shows that in all trenches, except the rigid seabed, there was a difference in the pull up and lay down bending moments, indicating the presence of pipe/soil suction.

Comparisons of the lay down bending moments from the different trenches showed which strain gauge locations can be compared directly. The reason for this was that changing the trench shape/depth changed the catenary shape of the riser and moved the TDP. This indicated that although the distance along the riser from the actuator had not changed, the distance of the strain gauge location to the TDP did. Comparing the difference between the bending moments at -0.8m and 1.4m actuator positions in Figure 4.21 shows that the naturally forming trench could be compared directly with the rigid seabed and that the artificially deepened trench could be compared directly with the backfilled trench.

The decrease in the bending moments due to the presence of pipe/soil suction (i.e. the change in bending moment during a pull up test) was examined. The increase in bending moment magnitude was approximately 11.3kNm, 10.7kNm and 10.5kNm (Figure 4.21) for the naturally forming trench, the artificially deepened trench and the back filled trench respectively. These values were similar (accounting for the shift in the strain gauges from the TDP) and indicated that the trench type had a small affect on the pipe/soil suction force.
Comparison of Bending Moments At Strain Gauge D for All Test Corridors

Figure 4.21 – Comparison of Bending Moments From All Trench Types

4.4.4 Observations on Pipe/Soil Suction

The test data from the harbour riser was examined in detail. A summary of the observations made with the evidence from the test data on the following topics are given below.

- Repeated loading
- Pull up velocity
- Consolidation time
- Suction release
- Suction kick

The bending moment response of strain gauge location D during a first pull up (test 3-5), a sixth pull up (test 3-5E) and an associated lay down (test 3-6) is shown in Figure 4.22. These show that after the first pull up test pipe/soil suction increases the magnitude of the bending moment peak by 85% (from -6.6kNm to -12.2kNm). However, for the sixth pull up the peak bending moment increase drops to 20% (-7.7kNm). This shows a 76% reduction in the bending moment response, and indicates that the pipe/soil suction force has reduced between the first and sixth pull
up tests. Figure 4.23 shows a summary of the minimum bending moments from pull up test series 3-5 compared to lay down test 3-6. It is shown that the pipe/soil suction force reduces by 66% between the first and second pull up tests, and then reduces further by around 4% for each subsequent test.

Degradation of Soil Suction with Sequential Pull Up Tests
Tests 3-5, 3-5E, 3-6

![Graph showing bending moments and actuator position](image)

**Figure 4.22 – Comparison of a First Pull Up Test with a Subsequent Pull Up Test**

Degradation of Soil Suction with Repeated Loading
Average Water Depth 2.5m
Pull Up Test Series 3-5, Lay Down Test 3-6

![Graph showing ratio of bending moments](image)

**Figure 4.23 – Effect of Repeated Loading on Bending Moment Response**

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Consecutive pull up tests 2-1C (third pull up) and 2-1D (fourth pull up) were conducted after repeated loading with pull up velocities of 0.1m/s and 0.01m/s, respectively. The results given in Figure 4.24 show that on remoulded clay pull up velocity has little effect on the bending moment response.

![Comparison of 2 Pull Up Tests with Different Pull Up Velocities](image)

**Figure 4.24 – Comparison of Pull Up Velocities**

The effect of consolidation time on strain gauge positions C and D during pull up tests 3-3 (4 hours consolidation) and 3-5 (12 hours consolidation) is shown in Figure 4.25. With increased consolidation time the magnitude of the bending moment response at strain gauge location C increases by 3kNm (58%) and at location D by 2kNm (23%).
After a pull up test actuation was complete (the pipe had been pulled to the top of the actuator) the bending moment response at strain gauges J and K was seen to continue to change. This is shown in Figure 4.26 where the bending moment response of strain gauge locations C, J and K and the corresponding actuator position are plotted with time. The vertical blue lines show the start and end of the pull up test. It can be seen that the bending moments did not change over the 10s before the pull up test starts. Once the test began all strain gauge locations showed a bending moment response similar to those previously observed. After the tests had finished the bending moment response at strain gauge C remained constant. However the bending moment response of strain gauges J and K continued to change for 15s and 18s respectively. This indicates that if a riser was left statically after pipe/soil suction had been mobilised the suction slowly dissipated and the riser moved into the equilibrium state, which had little or no pipe/soil suction.
The bending moment response of fast pull test 3-5, conducted with a sea level of 2.6m is shown in Figure 4.27. It can be seen that when the actuator moved past 0.6m the bending moment responses of strain gauges A, C, D and J started to oscillate. This appears to be due to a rapid release of pipe/soil suction, and is termed a suction kick.
4.4.5 Vertical Cyclic Tests

The cyclic tests were conducted to examine the effect of pipe/soil suction on the bending moments of a dynamically moving riser. The top of the harbour test riser was moved by the actuator to represent both day-to-day and extreme storm motions on the TDP. The top tensions and bending moments recorded during the extreme storm motions with the harbour test riser in the open trench (corridor 2) are shown in Figure 4.28 and Figure 4.29 respectively. The graphs show that over the ±0.4m actuator range there is little difference between the pull up and lay down bending moments. This indicates that during dynamic motions pipe/soil suction has a low effect on the bending moments. Examination of the bending moments during day-to-day motions also shows that the effect of pipe/soil suction was negligible.

Figure 4.28 – Top Tension Under Dynamic Motions at Nominal Actuator Position When Tested in the Open Trench in Water
Bending Moment with Actuator Motion,
Test 2-6 Dynamic Motions at Nominal, Open Trench in Water

Figure 4.29 – Bending Moment Under Dynamic Motions at Nominal Actuator Position When Tested in the Open Trench in Water

4.4.6 Observations of the Harbour Test Riser Trench

4.4.7 Trench Development

When the harbour test riser was installed care was taken to ensure that the seabed was left untouched. The riser was floated out at high tide, and then as the tide went out the riser was gently lowered onto the seabed. The top end of the riser was then lifted up to the actuator using a crane, and then pulled to meet the top cable using a winch. This process of laying the riser on the seabed, and then pulling in to the actuator mimics the offshore SCR installation process and shows the development of the SCR trench.

After the harbour test riser was installed the trench observed was 0.5 diameters deep and close fitting along the length of the riser as shown in Figure 4.30. In the catenary zone, where the harbour test riser has been lifted clear of the soil, an imprint of the riser can clearly be seen as shown in Figure 4.31.
Figure 4.30 – Close Fitting Trench Near TDP, After Riser was Installed

Figure 4.31 – Harbour Test Riser Trench Shortly After Installation in the Catenary Zone
4.4.8 Examination of Developed Trench

During the testing the trench was observed to deepen and widen around the TDP. Two photographs of the naturally formed trench are given in Figure 4.32A & B, Figure 4.32A looks from the TDP towards the actuator and Figure 4.32B looks from the TDP towards the anchor. These show the section of the harbour tests riser as it passed from the catenary zone, through the TDP into the buried zone and then into the surface zone where the pipe was connected to the anchor. The trench formed starts where the riser first touched the soil when the actuator was at its lowest position, which it was between most tests. The trench extends towards the anchor and the width increases from one diameter to a maximum of 2.5 diameters over a distance of 20m. The trench then reduces in width to one diameter over the next 40m at which point it was considered to be a static pipeline in the surface zone.

Figure 4.32A & B – Harbour Test Riser in Naturally Occurring Trench at Low Tide
Two close ups of the trench are shown in Figure 4.33. Both photographs were taken from the widest part of the trench; Figure 4.33A faces the anchor and the surface zone while Figure 4.33B faces the actuator and the catenary zone. The photographs show that there is no build up of soil around the top of the trench, which may be expected if the riser had been pushed into the trench walls by the tidal currents. It can also be seen that the tops of the trench wall were curved, indicating that the tidal currents could have eroded and smoothed the trench lips.

![Photograph A & B - Close up photographs of the trench at low tide](image)

**Figure 4.33A & B – Close up photographs of the trench at low tide**

### 4.4.9 Measurements

Measurements taken during the testing program given in Figure 4.34 and Figure 4.35 show the trench to be ladle shaped. The maximum depth and width increased over the six week testing period from 0.5 diameters to 1.2 diameters and from one diameter to 2.5 diameters respectively.
Figure 4.34 – Measurements of Trench Depth

Figure 4.35 – Measurements of Trench Width
4.4.10 Observations of Trenching from Cyclic Motions

An example of the trenching mechanism observed during the Watchet Harbour tests is shown in the photographs in Figure 4.36. These show a section of the harbour test riser that the TDP travels through during the extreme storm motion simulations. The actuator is towards the bottom left of the photographs and the anchor towards the top right. A description of the photographs follows:

- Photograph A shows the riser rested on the bottom of the water filled open trench. The water level was near the trench lips. The TDP was nearest the actuator.
- Photograph B shows the TDP moving through this section of the riser. As the riser was lifted out of the trench the water flows towards the anchor. Water could be seen in the bottom of the trench.
- Photograph C shows the section of the harbour test riser near the top of the extreme storm motion. All of the water had flowed towards the anchor.
- Photograph D shows the riser being lowered back into the trench. The water surged back into this section of the trench.
- Photograph E shows the riser near the end of the extreme storm motion, lying in the bottom of the trench. The water was still observed to be flowing towards the actuator.
- Photograph F shows the riser at the end of the dynamic cycle and in a similar position to that shown in Photograph A.

This series of photographs indicates two possible trenching mechanisms. These are:

- Vessel motions cause repeated loading on the trench floor that increases the penetration depth.
- Water pumped through the trench by TDP motions, eroding the trench walls and washing sediment out of the trench.
4.5 Harbour Test Riser Modelling

4.5.1 Introduction

An analytical model of the harbour test riser was produced to calibrate a finite element (FE) pipe/soil suction model. The pipe/soil interaction model used was created by the writer from a series of coarse 2D pipe/soil interaction tests conducted in STRIDE phase 2 and detailed in 2H Offshore (1999b) and Willis & West (2001). Details of the 2D pipe/soil interaction tests are not given here as the work conducted...
by the writer using an improved 2D pipe/soil interaction test apparatus is covered in the next chapter.

The FE analysis was conducted using the ANSYS (ANSYS Inc, 2000) code. The analysis of the harbour test riser reported here was originally conducted by the writer as part of the STRIDE phase 3 scope of work, 2H Offshore (2001a), and is detailed by 2H Offshore (2001b) and Bridge & Willis (2002).

4.5.2 Harbour Test Riser

The analytical harbour test riser model was created to be equivalent to the final harbour test riser as closely as possible. The model dimensions were taken from surveys of the riser and the seabed profile conducted during the testing program. Details of these dimensions are given in Table 4.1 and Figure 4.8. The analytical model for each analytical test corridor was then calibrated to the 'as built' riser by changing the length of the top cable. This changed the model top tension, which was then changed to correspond to the measured top tension of the harbour test riser.

4.5.3 Pipe/Soil Suction Model

The empirical pipe/soil suction curve used in the analysis was derived by the writer from the preliminary work on pipe/soil suction conducted during STRIDE Phase 2 and detailed by 2H Offshore (1999b) and 2H Offshore (2001b). The test parameters of the pipe/soil interaction test were chosen to correspond to the conditions of the harbour test riser that are given in Table 4.12. The pipe/soil suction curve, which is shown in Figure 4.37, consists of three sections: suction mobilisation, the suction plateau and suction release and are described below.

- Suction Mobilisation – As the riser initially moves upwards the pipe/soil suction force increases from zero to the maximum value.
- Suction Plateau – The pipe/soil suction force remains constant as the riser moved upwards
- Suction Release – Under further upward movement the pipe/soil suction force reduces from its maximum to zero at the break out displacement.
### Table 4.12 – Pipe/Soil Suction Model Properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pull Up Velocity</td>
<td>0.1 m/s</td>
</tr>
<tr>
<td>Consolidation Time, t</td>
<td>16 hours</td>
</tr>
<tr>
<td>Consolidation Load, $F_C$</td>
<td>64 kg</td>
</tr>
<tr>
<td>Maximum Pipe/soil Suction Force</td>
<td>812 N/m</td>
</tr>
<tr>
<td>Break-out Displacement</td>
<td>0.122 m</td>
</tr>
</tbody>
</table>

2D Pipe/Soil Suction Curves
From STRIDE II 2D Pipe/Soil Interaction Tests on Watchet Harbour Clay

![Suction Curves](image)

**Figure 4.37 – Pipe/Soil Suction Model, Willis and West (2001)**

#### 4.5.4 Analysis of Harbour Test Riser

Static analysis of the harbour test riser was conducted to calibrate the analytical models to the harbour test riser. Calibration of the analytical models was conducted by changing the length of the tuning chain so that the measured harbour test riser top tension during a lay down test (which was observed to exhibit no pipe/soil suction effects) corresponded to those calculated by the analytical model. A summary of the nominal tension and the tuning chain lengths obtained is shown in Table 4.13.
Table 4.13 – Length of Tuning Chain for Each Test Corridor Model

<table>
<thead>
<tr>
<th>Test Corridor</th>
<th>Nominal Top Tension (kN)</th>
<th>Tuning Chain Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 – Natural Trench</td>
<td>75.3</td>
<td>3.675</td>
</tr>
<tr>
<td>3 – Artificially Deepened Trench</td>
<td>69.7</td>
<td>3.68</td>
</tr>
<tr>
<td>4 – Back Filled Trench</td>
<td>69.7</td>
<td>3.68</td>
</tr>
<tr>
<td>5 – Rigid Seabed</td>
<td>90.6</td>
<td>3.61</td>
</tr>
</tbody>
</table>

Envelopes of the elevations, tension and bending moments along the harbour test risers length for test corridor 2 calculated using the analytical model during a no pipe/soil suction pull up test are shown in Figure 4.38, Figure 4.39 and Figure 4.40 respectively. The green lines on the graphs show the strain gauge locations.

The profile envelope given in Figure 4.38 shows the change in elevation of the riser and the horizontal distance that the TDP moves (approximately 28m) during pull up and lay down tests. The figure also shows the profiled seabed used in the analysis model and the effect that this has on the profile of the model.

The bending moment envelope given in Figure 4.39 shows the bending moment range along the length of the harbour test riser. It shows that near strain gauge location A the change in bending moment is expected to be small, and that strain gauges B, C, D, E, F, G, and H which are nearer the TDP will have bending moment ranges in the region of 12kNm. The bending moments at strain gauges I and J are calculated to be low, with a range of less than 5kNm and the bending moments at strain gauge locations K, L and M are calculated to be zero. These calculations are similar to those observed during the lay down tests that are given in Figure 4.20 and confirm that the analytical model is calculating realistic results.

The tension envelope along the riser length is given in Figure 4.40 and shows that the top tension changes between 50.9kN and 111.8kN, which correspond to the values
measured on the harbour test riser. This is expected as top tension was used to calibrate the analytical model to the test data.

Figure 4.38 – Profile Envelope along the Harbour Test Riser Model

Figure 4.39 – Bending Moments Envelope along Harbour Test Riser Model
4.6 Comparison of Experimental and Analytical Results

4.6.1 Rigid Seabed Tests

The first harbour test riser test data compared to the analytical model calculations were those from the rigid seabed tests. This test was chosen as there was no observed change during the pull up and lay down tests, and are assumed to be free from the influence of pipe/soil suction.

The calibration between the analytical and measured harbour test riser top tensions on a rigid seabed (test corridor 5) is given in Figure 4.41. This shows that the top tension from the analytical model coincides with the measured data from the pull up and lay down tests well and indicates that the analytical model was calibrated correctly for this test corridor.
Comparisons between results obtained using the analytical model and the harbour test riser are conducted using two methods. The first method compares the bending moment with actuator position from a single strain gauge location to that of a similar point on the analytical model while the second compares the calculated bending moment envelope with the overall change in bending moment.

The comparison between the bending moments from the analytical model and those measured from the harbour test riser at strain gauge location D are given in Figure 4.42. This shows that the analytical bending moment compares well to the measured data, both having a maximum bending moment close to zero at an actuator offset of -0.5m below nominal actuator position, and a minimum bending moment at an actuator offset of 1.4m above nominal actuator position of -8.65kNm and -8.66kNm for the measured and analytical data respectively. The error in the calculated bending moment range from the analytical model to the measured harbour test riser data is below 1%.
The measured changes in bending moments from the strain gauges on the harbour test riser were compared with the analytical bending moment envelopes. Analytical near bending moments were used as reference for the measured changes in bending moments. The positions along the riser of the strain gauge locations were not altered.

The analytical bending moment envelopes were in good agreement with the measured changes in bending moment. The calculations of bending moments at strain gauge locations A, D, K and M were good. However the bending moment range at strain gauge C during the lay down test was different to the bending moment range measured during the pull up test. Examination of the harbour test riser bending moment timetraces suggests that this strain gauge had started to fail during the lay down test, hence the difference in measure bending moment ranges.
4.6.2 Pull Up Tests

The calibration between the analytical and measured harbour test riser top tensions for test corridor 2, harbour test riser in a natural trench, and test corridor 4, harbour test riser in backfilled trench are given in Figure 4.44 and Figure 4.45 respectively. These show that the top tensions from the analytical models with and without pipe/soil suction match the measured data from the pull up and lay down tests well and indicate that the analytical model was calibrated correctly for these test corridors.

Figure 4.43 – Comparison of Analytical Bending Moment Envelope with Rigid Seabed Test Data
Comparisons between results obtained using the analytical model and the harbour test riser are conducted using two methods. The first compares the test data from a single strain gauge location to that of a similar point on the analytical model. The
magnitudes of the bending moments at the start of the analytical model are matched
to those of the harbour test riser to account for the effects of the uneven seabed. The
analysis using no pipe/soil suction (green line) shown in Figure 4.46 is compared to
the lay down test (blue line). The analysis using the upper bound soil curve (black
line) is compared to the pull up test (red line). Comparisons between the results from
the analytical modelling and test corridors 2 and 4 are shown in Figure 4.46 and
Figure 4.47 respectively. It can be seen that the analytical model using the upper
bound pipe/soil suction curve predicts the test data well.

Figure 4.46 – Comparison of Test Data with Analytical Model for Test Area 2
The second method of comparing measured and analytical results is achieved using the bending moment envelopes from the analytical calculations for the no pipe/soil suction and with suction models. These are compared to the maximum and minimum bending moments measured during pull up and lay down tests, as shown in Figure 4.48. As before, the green and black lines represent the no pipe/soil suction and the upper bound pipe/soil suction models respectively (Note that the top most black line is on top of the green line). The red and blue lines show the minimum and maximum bending moments from pull up (with pipe/soil suction) and lay down (no pipe/soil suction) tests. The difference between the red and blue lines is the effect on the bending moment of the pipe/soil suction.

It can be seen that the analytical bending moment envelopes compare well to the test data ranges; the no pipe/soil suction model predicting the lay down test data ranges well. The effect of pipe/soil suction is also predicted well, with strain gauges C, D and F showing a similar response to the upper bound soil model. The analytical model also calculates the response of strain gauges J and K, which exhibit a lower change in bending moment during the pull up test than lay down test.
Further comparison conducted between the measured harbour test riser bending moments and the calculated analytical bending moments are included in Appendix C. These show similar observations, that the calibrated analytical models accurately reproduce the bending moments measured on the harbour test riser.

![Bending Moment Envelope, Top Cable Length 3.675m, Strain Gauge Data from Tests 2-10 and 2-11](image)

Figure 4.48 – Comparison of Test Data and Analytical Bending Moment Envelope

4.7 Discussion and Conclusions

4.7.1 Overview

The harbour test riser was a full-scale model of the TDZ of a SCR. The soil used in the tests was a marine sediment with similar properties to those of deepwater Gulf of Mexico clays. The tests conducted examine the effect of pipe/soil interaction on bending moments and tensions in the TDZ during slow drift, day-to-day dynamic and storm type motions. The harbour test riser also used a rigid seabed that provides benchmark data for validating SCR analysis techniques, such as finite element analysis and closed form solutions. Observations made during the tests also give insights into the trenching mechanisms that create the SCR trenches shown in the previous chapter.
4.7.2 Pipe/Soil Suction

The harbour test riser tests were conducted on four seabed types, including naturally formed trench, artificially deepened trench, backfilled and rigid seabed. The tests showed similar changes in bending moment at all strain gauge locations during lay down tests and the rigid seabed pull up tests. When the riser was initially in contact with the soil (which is all trenches except the rigid seabed) the pull up tests showed an increase in bending moment. When the riser was on the rigid seabed the pull up test bending moments were virtually identical to the lay down bending moments. This confirms that the difference in the bending moments is due to the bond between the soil and the pipe, which has been termed pipe/soil suction.

The effect of pipe/soil suction was observed to increase the maximum bending moment by up to a factor of two. This occurred during pull up tests that simulate either long period motions or vessel drift. This indicates that during long period motions the effects of pipe/soil suction on SCRs should be considered.

It was observed that during pull up tests, after the actuator had stopped moving the bending moment at strain gauge locations J and K would continue to change. This effect was not seen in the lay down tests, and therefore was concluded to be due to the pipe/soil suction force dissipating so that the TDP continued to peel away from the seabed and the riser moved into a static equilibrium position.

Examination of a sequence of pull up tests shows that the maximum bending moment reduces with every subsequent test in the series. After the first pull up test the maximum bending moment reduces by around 60%, then after the second test by a further 15%. After six consecutive tests the maximum bending moment was observed to be less that 20% of the bending moment observed during the first pull up test.

Similar comparisons were conducted with the day-to-day and extreme storm dynamic motions. The results of these tests show that the difference in bending moments near the TDP of the harbour test riser is minimal. This indicates that the pipe/soil suction force:
• Dissipates during repeated soil loading
• May not cause any significant increase in riser stress when the SCR is subject to wave induced vessel motions.

The tests also showed some effects of pipe/soil suction on the harbour test riser, including the suction kick and suction release where the pipe/soil suction force is suddenly and quickly released.

4.7.3 Trenches

The trenches observed in the harbour test riser are similar to those observed in the SCR trench surveys shown in the previous chapter. It was observed that the trench width and depth increased during the six weeks of testing. Initially the harbour test riser trench was close fitting with the riser penetrated to a depth of approximately 0.5 diameters. After six weeks of testing the general profile of a trench was ladle shaped, being deepest near the TDP, then reducing in depth as you move along the riser towards the anchor. In plan the trench was observed to be close fitting near the anchor and up to 3.0 diameters wide nearest the TDP. Near the actuator the trench reduced in width to the imprint made during installation of the harbour test riser.

Observations made during the testing gave insights into SCR trench mechanisms that include:

• The dynamic motions applied by the actuator, representing the vessel motions, caused the riser to dig into the seabed forming the trench.
• Any vertical motion at the TDP caused the water beneath the riser to be pumped out of the trench, carrying sediment with it.

It is also surmised that when the harbour test riser was submerged the buoyancy force caused the riser to lift away from the seabed. Any lose sediment that was in the trench or attached to the riser was washed away.
4.7.4 Critical Assessment of Tests

The full scale tests provide a valuable data set for validating small scale and FE models. The main source of data from the tests was bending moment values at the strain gauge location and top and anchor tension as these are considered to be the most significant forces within the TDZ. However additional data, such as the soil pressure would have provided a more complete picture of the full scale pipe/soil interaction.

The test matrix completed during the harbour tests was comprehensive, examining slow drift and dynamic motions. However as in all experiments of this nature, analysis of the data reveals additional tests that could have been conducted, for example the pull up speed could have been varied to see how the pipe/soil suction force varies with pull up velocity.

4.7.5 Analysis of Harbour Test Riser

The analytical modelling of the harbour test riser shows that a FE model calibrated with the measured top tension can be used to accurately calculate bending moments along the length of the harbour test riser. An empirical pipe/soil suction model was derived by the writer from earlier pipe/soil interaction tests conducted within the STRIDE JIP. The parameters of the 2D test, such as pipe pull up speed and trench depth, were matched to those of the harbour test riser and the derived curve used in the analytical model to calculate the effect of pipe/soil suction on harbour test riser bending moments. The analysis shows that a pipe/soil suction model can be accurately used to model the affects on the bending moment response of the harbour test riser due to the pipe/soil suction force. However the empirical pipe/soil suction model is only applicable to the harbour test riser. Further investigation is required to derive pipe/soil suction models that are more widely applicable to SCR analysis.
5.0 2D PIPE/SOIL SUCTION TESTS

5.1 Introduction

The STRIDE small scale 2D pipe/soil interaction tests were conducted to examine the force generated in the pipe-soil interface, when a submerged pipe is pulled from a trench in soft marine clay. The test rig used in these experiments, shown in Figure 5.1, consisted of a pipe suspended over a test tank containing a sample of Watchet Harbour clay (a sediment with similar properties to the deepwater clays found in the Gulf of Mexico). The 2D test rig represented a section of a steel catenary riser (SCR) within the touchdown zone (TDZ). The vertical actuations were similar in velocity and amplitude to the typical wave and slow drift motions that would occur at the touchdown point (TDP) on a SCR.

![Figure 5.1 – Photographs of the STRIDE 2D Test Rig](image)

The objectives of this testing campaign were to examine the effects of pull-out velocity, consolidation time, consolidation load and pipe diameter on the force/displacement curve, generated when a pipe (or riser) is pulled out of a trench.
The intention was that this data could then be used to develop a comprehensive numerical model for use in SCR analysis.

5.2 Design of Test Rig

5.2.1 STRIDE Test Pipes

During the testing two pipes were used, a larger diameter glass reinforced plastic pipe and a small diameter steel pipe, the properties of which are summarised in Table 5.1. The pipes used in the test both had an aspect ratio (length/diameter) of over 4, which indicated that the end effects from the pipes were small and therefore assumed to be negligible, Marintek (2000).

<table>
<thead>
<tr>
<th>Pipe Parameter</th>
<th>Large Diameter Pipe</th>
<th>Small Diameter Pipe</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>0.1683m (6-5/8&quot;)</td>
<td>0.0484m (1.9&quot;)</td>
</tr>
<tr>
<td>Length</td>
<td>0.948m (37.3&quot;)</td>
<td>0.948m (37.3&quot;)</td>
</tr>
<tr>
<td>Material</td>
<td>Glass Reinforced Plastic (GRP)</td>
<td>Steel</td>
</tr>
<tr>
<td>In Air Pipe Weight</td>
<td>37.5kg</td>
<td>13.8kg</td>
</tr>
<tr>
<td>Submerged Pipe Weight</td>
<td>15.3kg (33.7lb)</td>
<td>12.0kg (26.4lb)</td>
</tr>
<tr>
<td>Aspect Ratio</td>
<td>5.6</td>
<td>19.6</td>
</tr>
</tbody>
</table>

5.2.2 STRIDE Test Rig Properties

The pipe was actuated using a pulley system, allowing the pipe to be lowered into the clay and subsequently pulled out. The pull-out velocity of the pipes was controlled by matching the pulley hand crank rate to an electronic metronome. Uplift resistance was measured using a load cell, connected in series between the pulley system and the pipe. Pipe position was measured using an ultra-sonic displacement sensor. The load cell and the ultra-sonic displacement sensors were connected to a PC that logged the measurements using InstruNet (GWI, 1998) digital to analogue converters and the DASYLab logger software by DASYTEC (1996). The positions on the test rig of
the load cells and the ultra-sonic displacement sensor are shown in Figure 5.2. A summary of the test rig properties is given in Table 5.2. The test rig was sized so that the large and small diameter pipes were completely submerged in the water and that the soil remained saturated. Details of the load cell and the ultrasonic displacement sensor are given in Table 5.3.

![Figure 5.2 – Schematic of Experimental Set up](image)

**Table 5.2 – Test Rig Properties**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of Tank</td>
<td>1.075m</td>
</tr>
<tr>
<td>Width of Tank</td>
<td>0.500m</td>
</tr>
<tr>
<td>Depth of Tank</td>
<td>0.576m</td>
</tr>
<tr>
<td>Depth of Clay</td>
<td>0.338m</td>
</tr>
<tr>
<td>Depth of Water</td>
<td>0.133m</td>
</tr>
</tbody>
</table>

**5.2.3 Instrument Calibration**

The load cell (model DBBSE-100kg) and the displacement sensor (model Analog Q45U) were both calibrated. A known load was hung from the load cell and
measurements were taken using DASYLab. The calibration factors within DASYLab were then altered so that the measurement was the same as the known load. The displacement sensor was calibrated against standard measured distances within the test rig. These were compared with the DASYLab reading, and the ultra sonic sensor was calibrated so that the electronic measurements matched the standard measurements.

Table 5.3 – Summary of Sensor Properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Load Cell</th>
<th>Ultra Sonic Displacement Sensor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>DBBSE-100kg</td>
<td>Analog Q45U</td>
</tr>
<tr>
<td>Range</td>
<td>0 – 100kg</td>
<td>0.25 – 3.0m</td>
</tr>
<tr>
<td>Response Time</td>
<td>-</td>
<td>80 ms</td>
</tr>
<tr>
<td>Supply voltage</td>
<td>10V</td>
<td>15 – 24V</td>
</tr>
<tr>
<td>Compensated Operating</td>
<td>-</td>
<td>0 – 50°</td>
</tr>
<tr>
<td>Temperature Effect</td>
<td>&lt;0.0015%/°C</td>
<td>-</td>
</tr>
</tbody>
</table>

5.2.4 Soil Preparation

The clay soil was transported from Watchet Harbour, already placed in the test tank, in October 1999. The clay soil was reconstituted and then left to consolidate for 15 months with a surface load applied for the first three months. Because of the length of the consolidation time the clay was considered to be in an undisturbed state for testing purposes.

5.2.5 Geotechnical Data

The general properties of the Watchet Harbour clay are summarised in Table 5.5 and were found to be similar to those of the high plasticity clays found in deepwater Gulf of Mexico. The undrained shear strength, which is dependent on the loading history, was measured using a shear vane at varying plan positions and depths within the test tank. This data was found to be consistent and a shear strength profile was developed, Table 5.4 and Figure 5.3.
Table 5.4 – Undrained Shear Strength at ½ Diameter Depth

<table>
<thead>
<tr>
<th>Shear Strength, $S_u$</th>
<th>0.0484m Pipe</th>
<th>0.1683m Pipe</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undisturbed Shear Strength at ½D</td>
<td>1.08 kPa</td>
<td>2.86 kPa</td>
</tr>
<tr>
<td>Remoulded Shear Strength at ½D</td>
<td>0.33 kPa</td>
<td>0.85 kPa</td>
</tr>
<tr>
<td>Sensitivity of Clay at ½D</td>
<td>3.3</td>
<td>3.3</td>
</tr>
</tbody>
</table>

Table 5.5 – Soil Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content, $w$</td>
<td>104.7%</td>
</tr>
<tr>
<td>Bulk Density, $\rho$</td>
<td>1.46 Mg/m³</td>
</tr>
<tr>
<td>Dry Density, $\rho_d$</td>
<td>0.73 Mg/m³</td>
</tr>
<tr>
<td>Particle Density, $\rho_s$</td>
<td>2.68 Mg/m³</td>
</tr>
<tr>
<td>Liquid Limit, $w_L$</td>
<td>87.6%</td>
</tr>
<tr>
<td>Plastic Limit, $w_P$</td>
<td>38.8%</td>
</tr>
<tr>
<td>Plasticity Index, $I_p$</td>
<td>48.9%</td>
</tr>
<tr>
<td>Average Organic Content</td>
<td>3.2%</td>
</tr>
<tr>
<td>Specific Gravity, $G_s$</td>
<td>2.68</td>
</tr>
<tr>
<td>Coefficient of Consolidation, $c_v$ at ½D</td>
<td>0.5 m²/year</td>
</tr>
<tr>
<td>Coefficient of Volume Compressibility, $m_v$ at ½D</td>
<td>15 m²/MN</td>
</tr>
<tr>
<td>Undisturbed Shear Strength at ½D</td>
<td>2.86 kPa</td>
</tr>
<tr>
<td>Remoulded Shear Strength at ½D</td>
<td>0.85 kPa</td>
</tr>
<tr>
<td>Sensitivity of Clay at ½D</td>
<td>3.3</td>
</tr>
</tbody>
</table>

Notes
1 Diameter is defined as 0.1683m
Figure 5.3 – Undrained Shear Strength of Watchet Clay in Test Tank

5.3 Test Method

5.3.1 Test Procedure

Each pull-out test was conducted in a similar manner. The following outlines the test procedure.

- The pipe was lowered slowly into the trench. The pipe was steadied so that the interference with the trench wall due to the pipe twisting was minimal.
- On short consolidation tests the pipe was left for five minutes, on tests with a longer consolidation time and a consolidation load a cradle was attached to the top of the pipe into which weights were placed.
- After the consolidation time was over the cradle and weights (if any) were removed and the pull-out test was conducted. The rotational speed of the hand crank pulley was matched to the signal of an electronic metronome to obtain a constant pull-out velocity; a number of rotations of the hand crank was required to take in the slack in the wires. Once the pull-out had started it was not stopped until the pipe was clear of the trench.
- The load timetrace data was corrected for pipe weight and converted into force by multiplying by the acceleration due to gravity.
• The displacement data was altered so that zero displacement occurred with zero force (where the wires were supporting only the pipe weight, and without uplift resistance).
• The pull-out velocity was calculated from the gradient of the displacement time trace during the pull-out test.
• The test data was filtered using a low pass filter to remove any noise.
• Each test was repeated at least twice to ensure the consistency of the results.

5.3.2 Trenches
The trenches used in the tests were created in the same manner; a pipe was pushed into the soil to a depth of half diameter and left under a 63.6kg consolidation load for three days. The pipe was then pulled out, and placed back into the soil. This forms a close fitting, but loose trench where the sides of the trench did not completely hug the pipe for the full half diameter depth, Figure 5.4. Most tests reported here were conducted in a loose trench as this was assumed to be similar to those reported from SCR surveys. Additional tests were conducted in a tight trench where additional soil was added around the sides of the pipe, representing trench backfill, so that the clay was in contact with the pipe for the embedded circumference.

![Figure 5.4 - Tight and Loose Trenches](image)

5.3.3 Normalisation Analysis
The comparisons of test data were conducted using normalised test data to account for the different pipe diameter, pull-out speed, consolidation time and consolidation loads used in the tests. The equations used are given below.
Uplift resistance force (dimensionless), was derived from bearing capacity theory and comparable with N as given by Byrne and Finn (1978) is found from:

\[ N = \frac{Q_s}{L D S_U} \]  

(5.1)

where

- \( D \) is the external diameter of the pipe
- \( L \) is the length of the pipe
- \( Q_s \) is the maximum uplift resistance force
- \( S_U \) is the soil shear strength at depth \( z \)

The uplift displacement, \( \Delta \), trench depth, \( z \), and pull-out velocity, \( V \), were all normalised using the pipe diameter. The consolidation load, \( F_c \), was normalised by dividing the load by the pipe length, \( L \), and pipe diameter. The consolidation time was normalised using an equation derived from the equation for the average degree of consolidation, \( U \) from settlement and consolidation theory.

\[ \sqrt{\frac{c_v t}{D^2}} \]  

(5.2)

where

- \( c_v \) is the coefficient of consolidation
- \( t \) is the consolidation time

The normalisation methods for consolidation time and load were then multiplied together to form a combined normalisation equation, which is given below and has dimensions of \( F/L^2 \), or SI units of \( N/m^2 \).

\[ \frac{F_c}{L D} \times \sqrt{\frac{c_v t}{D^2}} \]  

(5.3)

Vesic (1971) stated that any computations of break out forces should be based on an estimate of the soil strength at the time of break out. The shear strength used in the normalisation depended on the consolidation time. If the consolidation time was low, less than five minutes, then the remoulded shear strength was used but in all other cases the undisturbed shear strength was used.
5.4 Test Program

Tests were conducted to examine the uplift resistance curve and then the effects of pipe diameter, pull-out velocity, consolidation time and the consolidation load on the pipe/soil suction curve, Table 5.6. The repeatability tests were designed to show that the results from similar tests were comparable and that hand actuation of the pulley produced consistent results. The final set of tests was conducted to compare with the CARISIMA tests.

Table 5.6 – STRIDE 2D Vertical Pull-Out Test Matrix

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Pipe Diameter (m)</th>
<th>Pull-Out Velocity (m/s)</th>
<th>Consolidation Time (min or Hrs)</th>
<th>Consolidation Load (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Repeatability of Tests</td>
<td>0.1683</td>
<td>0.012</td>
<td>5min</td>
<td>0.0</td>
</tr>
<tr>
<td>Pull-Out Velocity, 1</td>
<td>0.1683</td>
<td>0.0015, 0.003, 0.006, 0.012</td>
<td>5min</td>
<td>0.0</td>
</tr>
<tr>
<td>Consolidation Time</td>
<td>0.1683</td>
<td>0.006</td>
<td>5min, ½, 1, 2, 4, 8, 16 and &gt;72hours</td>
<td>63.6</td>
</tr>
<tr>
<td>Consolidation Load</td>
<td>0.1683</td>
<td>0.006</td>
<td>5min</td>
<td>0.0, 63.6</td>
</tr>
<tr>
<td>Pull-Out Velocity, 2</td>
<td>0.0484</td>
<td>0.003, 0.006, 0.012</td>
<td>5min</td>
<td>0.0</td>
</tr>
<tr>
<td>Pipe in Tight Trench</td>
<td>0.0484</td>
<td>0.003, 0.006, 0.01, 0.012, 0.017</td>
<td>5min</td>
<td>0.0</td>
</tr>
<tr>
<td>CARISIMA Comparison</td>
<td>0.0484</td>
<td>0.01</td>
<td>16 hrs</td>
<td>0.0</td>
</tr>
</tbody>
</table>

5.5 Experimental Results and Observations

5.5.1 General Observations

The force and displacement data from the 2D vertical pull-out tests can be combined to produce a pipe/soil suction curve, an example of which is shown in Figure 5.5.
The pull-out force/displacement curve starts at the origin, then increases to a maximum force, termed the maximum uplift resistance force over a small range of displacement, typically less than 1mm. The force remains reasonably constant as the displacement increases. This section of the curve is termed the suction plateau. After a distance the force decreases to zero and the pipe loses contact with the soil; the bond between the pipe and the soil is broken. This displacement is termed the breakout displacement, $\Delta_B$.

During the tests it was observed that one end of the pipe would break out first, and the failure between the pipe and the soil would ‘unzip’ or ‘peel’ from one end of the pipe to the other. This was possible as the pipe was connected to the pulley wire via two strops, one at each end of the pipe. It was also noted that either end of the pipe could initiate the pipe peeling. The peeling effect observed is consistent with observations from the full scale pull-out tests using a catenary riser (described in Chapter 4 of this thesis).

5.5.2 Repeatability of the Tests

Initial experiments were conducted to examine the repeatability of a set of four similar tests. The pipe was lowered into the pre-formed trench and left for five
minutes. The pipe was then pulled-out of the trench at a constant velocity. The variation of pipe position with time are shown in Figure 5.6, illustrating consistent pull-out velocity. The corresponding pull-out curves are shown in Figure 5.7 with uplift force in the vertical axis and pull-out displacement in the horizontal axis. Each of the tests has a similar trend with similar values of uplift resistance force and displacement, indicating that the experiments had good repeatability.

A summary of the maximum pull-out force, break-out displacement and pull-out velocity are given in Table 5.7. The mean and standard deviation for each of the parameters is calculated and given in the bottom two rows of the table. These values show that the spread of the test data is small relative with the mean and confirm that the repeatability of the experiments was good.

<table>
<thead>
<tr>
<th>Test</th>
<th>Pull-Out Velocity (mm/s)</th>
<th>Normalised Pull-out Velocity (D/s)</th>
<th>Maximum Uplift Force (N)</th>
<th>Normalised Maximum Uplift Force (-)</th>
<th>Break-out Displace'nt (m)</th>
<th>Normalised Break-out Displace'nt (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T14A</td>
<td>12.1</td>
<td>0.072</td>
<td>201.3</td>
<td>1.48</td>
<td>0.0138</td>
<td>0.082</td>
</tr>
<tr>
<td>T14B</td>
<td>11.1</td>
<td>0.066</td>
<td>180.6</td>
<td>1.33</td>
<td>0.0136</td>
<td>0.081</td>
</tr>
<tr>
<td>T14C</td>
<td>12.5</td>
<td>0.074</td>
<td>175.7</td>
<td>1.30</td>
<td>0.0141</td>
<td>0.084</td>
</tr>
<tr>
<td>T14D</td>
<td>11.3</td>
<td>0.067</td>
<td>192.7</td>
<td>1.42</td>
<td>0.0137</td>
<td>0.081</td>
</tr>
<tr>
<td>Mean</td>
<td>11.8</td>
<td>0.070</td>
<td>187.6</td>
<td>1.38</td>
<td>0.0138</td>
<td>0.082</td>
</tr>
<tr>
<td>Std Dev</td>
<td>0.661</td>
<td>0.0039</td>
<td>11.61</td>
<td>0.0826</td>
<td>0.0002</td>
<td>0.0014</td>
</tr>
</tbody>
</table>
5.5.3 Pull-Out Velocity

A series of pull-out tests was conducted with the 0.1683m outer diameter pipe to examine the effect of the pull-out velocity on the pipe/soil interaction curves. Four
pull-out velocities were considered, 1.5mm/s, 3.0mm/s, 6.0mm/s and 12.0mm/s. The test pipe was lowered into the existing loose trench and left to settle for five minutes, after which the test pipe was pulled out of the trench at a consistent pull-out velocity. This test format was repeated at least three times for each of the four pull-out velocities used. Comparisons of the first set of uplift resistance tests are shown in Figure 5.8 with a summary of the results in Table 5.8. Observations are made below:

- As the pull-out velocity increased the maximum uplift force also increased.
- The pull-out force typically increased by 50N for each doubling of the pull-out velocity.
- As the pull-out velocity increased the break-out displacement also increased.
- The break out displacement increased on average by 2.5 times as the pull-out velocity increased from 1.4mm/s to 12mm/s.
- A pull-out resistance curve for a given velocity did not overlap a pull-out resistance curve for a different velocity.
- The shape of the force/displacement curve was similar for all of the tests.
Table 5.8 – Summary of Pull-out Velocities and Maximum Uplift Force

<table>
<thead>
<tr>
<th>Test</th>
<th>Pull-Out Velocity (mm/s)</th>
<th>Normalised Pull-out Velocity (D/s)</th>
<th>Maximum Uplift Force (N)</th>
<th>Normalised Maximum Uplift Force (-)</th>
<th>Break-out Displace'nt (m)</th>
<th>Normalised Break-out Displace'nt (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T14A</td>
<td>12.1</td>
<td>0.072</td>
<td>201.3</td>
<td>1.48</td>
<td>0.0138</td>
<td>0.082</td>
</tr>
<tr>
<td>T14B</td>
<td>11.1</td>
<td>0.066</td>
<td>180.6</td>
<td>1.33</td>
<td>0.0136</td>
<td>0.081</td>
</tr>
<tr>
<td>T14C</td>
<td>12.5</td>
<td>0.074</td>
<td>175.7</td>
<td>1.30</td>
<td>0.0141</td>
<td>0.084</td>
</tr>
<tr>
<td>T14D</td>
<td>11.3</td>
<td>0.067</td>
<td>192.7</td>
<td>1.42</td>
<td>0.0137</td>
<td>0.081</td>
</tr>
<tr>
<td>T12A</td>
<td>6.3</td>
<td>0.037</td>
<td>170.4</td>
<td>1.26</td>
<td>0.0104</td>
<td>0.062</td>
</tr>
<tr>
<td>T12B</td>
<td>6.2</td>
<td>0.037</td>
<td>148.4</td>
<td>1.09</td>
<td>0.0109</td>
<td>0.065</td>
</tr>
<tr>
<td>T12C</td>
<td>6.5</td>
<td>0.039</td>
<td>145.7</td>
<td>1.07</td>
<td>0.0111</td>
<td>0.066</td>
</tr>
<tr>
<td>T12D</td>
<td>6.2</td>
<td>0.037</td>
<td>155.1</td>
<td>1.14</td>
<td>0.0122</td>
<td>0.072</td>
</tr>
<tr>
<td>T13A</td>
<td>3.0</td>
<td>0.018</td>
<td>91.5</td>
<td>0.67</td>
<td>0.0079</td>
<td>0.047</td>
</tr>
<tr>
<td>T13B</td>
<td>3.0</td>
<td>0.018</td>
<td>78.1</td>
<td>0.58</td>
<td>0.0083</td>
<td>0.049</td>
</tr>
<tr>
<td>T13C</td>
<td>3.0</td>
<td>0.018</td>
<td>89.1</td>
<td>0.66</td>
<td>0.0081</td>
<td>0.048</td>
</tr>
<tr>
<td>T16A</td>
<td>1.6</td>
<td>0.009</td>
<td>48.0</td>
<td>0.35</td>
<td>0.0053</td>
<td>0.031</td>
</tr>
<tr>
<td>T16B</td>
<td>1.4</td>
<td>0.008</td>
<td>43.3</td>
<td>0.32</td>
<td>0.0047</td>
<td>0.028</td>
</tr>
<tr>
<td>T16C</td>
<td>1.4</td>
<td>0.008</td>
<td>51.3</td>
<td>0.38</td>
<td>0.0067</td>
<td>0.040</td>
</tr>
</tbody>
</table>
Varying Pull Out Velocities
Unconsolidated Clay, Pipe Weight 15.3kg, Pre-load 0kg, Pipe Diameter 0.1683m, Pipe Length 0.948m

![Graph showing pull-out resistance variation with pull-out velocity](image)

**Figure 5.8 – Pull-out Resistance Variation with Pull-out Velocity**

The maximum uplift resistance verses pull-out velocity is given in Figure 5.9. A line of best fit was used to define an equation to calculate the normalised maximum uplift force if the normalised velocity is known. It was found that a power law relationship gave the best fit for the available data (correlation coefficient of 0.933) with a relationship of:

$$\frac{Q_s}{LDS_U} = 7.33 \left(\frac{V}{D}\right)^{0.60} \quad (5.6)$$

The breakout displacement, $\Delta_B$ versus the pull-out velocity is given in Figure 5.10. A line of best fit was used to determine an equation to calculate the normalised displacement if the normalised velocity is known. It was found that a power relationship gave the best fit for the available data (correlation coefficient of 0.944) with a relationship of:

$$\frac{\Delta_B}{D} = 0.26 \left(\frac{V}{D}\right)^{0.43} \quad (5.7)$$
5.5.4 Consolidation Time

A series of pull-out tests were conducted to examine the effect of the consolidation on the pipe/soil interaction curves. Eight consolidation times were compared; five
minutes, half hour, one hour, two hours, four hours, eight hours, sixteen hours and >72 hours. The test pipe was lowered into the trench and left to settle with a pre-load of 63.6kg, after which time the test pipe was pulled out of the trench at a consistent pull-out velocity of 6.0mm/s. This test format was repeated three times for each of the eight consolidation times used.

A comparison of all of the consolidation tests is shown in Figure 5.11 with selected tests shown in Figure 5.12. A summary of the results is given in Table 5.9. The main observations are as follows:

- Tests with low consolidation times had relatively low maximum uplift resistance force, five minutes consolidation time results in an average N of 0.45.
- Tests with high consolidation times had relatively high maximum uplift resistance force, 112 hours consolidation time results in a N of 0.89.
- With low consolidation times (five minutes, half hour) the maximum uplift forces were similar.
- Break-out displacement increased slightly as consolidation time increased. With ½ hour consolidation time the average normalised $\Delta_B$ was 0.116 which increases to 0.168 at 16 hours then decreases to 0.119 at 112 hours. This result was thought to be due to experimental error rather than consolidation time.
- The shape of the force/displacement curve was similar for all of the tests.
<table>
<thead>
<tr>
<th>Test</th>
<th>Consolidation Time (Hours)</th>
<th>Normalised Consolidation Load and Time (Pa)</th>
<th>Maximum Uplift Force (N)</th>
<th>Normalised Maximum Uplift Force (−)</th>
<th>Normalised Break-out Displacement (−)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24A-3</td>
<td>112</td>
<td>1120</td>
<td>406.9</td>
<td>0.89</td>
<td>0.119</td>
</tr>
<tr>
<td>24A-2</td>
<td>78.0</td>
<td>780</td>
<td>366.9</td>
<td>0.80</td>
<td>0.131</td>
</tr>
<tr>
<td>24A-1</td>
<td>16.5</td>
<td>165</td>
<td>355.8</td>
<td>0.78</td>
<td>0.168</td>
</tr>
<tr>
<td>24E-1</td>
<td>16.5</td>
<td>165</td>
<td>286.8</td>
<td>0.63</td>
<td>0.131</td>
</tr>
<tr>
<td>24E-2</td>
<td>16.0</td>
<td>160</td>
<td>313.2</td>
<td>0.69</td>
<td>0.111</td>
</tr>
<tr>
<td>24E-3</td>
<td>16.0</td>
<td>160</td>
<td>363.9</td>
<td>0.80</td>
<td>0.115</td>
</tr>
<tr>
<td>24E-4</td>
<td>16.0</td>
<td>160</td>
<td>398.1</td>
<td>0.87</td>
<td>0.135</td>
</tr>
<tr>
<td>24G-1</td>
<td>8.0</td>
<td>80</td>
<td>300.5</td>
<td>0.66</td>
<td>0.142</td>
</tr>
<tr>
<td>24G-2</td>
<td>8.0</td>
<td>80</td>
<td>357.3</td>
<td>0.78</td>
<td>0.147</td>
</tr>
<tr>
<td>24G-3</td>
<td>8.0</td>
<td>80</td>
<td>346.0</td>
<td>0.76</td>
<td>0.116</td>
</tr>
<tr>
<td>24F-1</td>
<td>4.0</td>
<td>40</td>
<td>252.8</td>
<td>0.55</td>
<td>0.108</td>
</tr>
<tr>
<td>24F-2</td>
<td>4.0</td>
<td>40</td>
<td>315.0</td>
<td>0.69</td>
<td>0.106</td>
</tr>
<tr>
<td>24F-3</td>
<td>4.0</td>
<td>40</td>
<td>295.2</td>
<td>0.65</td>
<td>0.099</td>
</tr>
<tr>
<td>24D-1</td>
<td>2.0</td>
<td>20</td>
<td>216.2</td>
<td>0.47</td>
<td>0.128</td>
</tr>
<tr>
<td>24D-2</td>
<td>2.0</td>
<td>20</td>
<td>254.2</td>
<td>0.56</td>
<td>0.133</td>
</tr>
<tr>
<td>24D-3</td>
<td>2.0</td>
<td>20</td>
<td>237.8</td>
<td>0.52</td>
<td>0.109</td>
</tr>
<tr>
<td>24C-1</td>
<td>1.0</td>
<td>10</td>
<td>213.4</td>
<td>0.47</td>
<td>0.118</td>
</tr>
<tr>
<td>24C-2</td>
<td>1.0</td>
<td>10</td>
<td>206.1</td>
<td>0.45</td>
<td>0.111</td>
</tr>
<tr>
<td>24C-3</td>
<td>1.0</td>
<td>10</td>
<td>216.6</td>
<td>0.47</td>
<td>0.112</td>
</tr>
<tr>
<td>24B-1</td>
<td>0.5</td>
<td>5.0</td>
<td>223.7</td>
<td>0.49</td>
<td>0.112</td>
</tr>
<tr>
<td>24B-2</td>
<td>0.5</td>
<td>5.0</td>
<td>209.0</td>
<td>0.46</td>
<td>0.113</td>
</tr>
<tr>
<td>24B-3</td>
<td>0.5</td>
<td>5.0</td>
<td>185.4</td>
<td>0.41</td>
<td>0.123</td>
</tr>
<tr>
<td>24H-1</td>
<td>0.1</td>
<td>1.0</td>
<td>219.6</td>
<td>0.48</td>
<td>0.071</td>
</tr>
<tr>
<td>24H-2</td>
<td>0.1</td>
<td>1.0</td>
<td>206.1</td>
<td>0.44</td>
<td>0.071</td>
</tr>
<tr>
<td>24H-3</td>
<td>0.1</td>
<td>1.0</td>
<td>220.5</td>
<td>0.48</td>
<td>0.071</td>
</tr>
</tbody>
</table>
Figure 5.11 – Effect of Increasing Consolidation Time on Uplift Force
Scatter plots showing the trend of normalised maximum uplift resistance against normalised consolidation time is given in Figure 5.13. The trend of normalised breakout displacement against normalised consolidation time is shown in Figure 5.14. A line of best fit was used to define a relationship between the normalised maximum uplift resistance force if the consolidation time and load was known. It was found that a power law gave the best fit (correlation coefficient 0.749) with a relationship of:

$$\frac{Q_s}{LDS_U} = 0.166 \left( \frac{F_C}{LD} \sqrt{\frac{C_p t}{D^2}} \right)^{0.22}$$  \hspace{1cm} (5.8)

The relationship between the normalised break out displacement and the normalised consolidation time was reasonable as the correlation coefficient was 0.454. However, the trend was best represented using a power law (correlation coefficient 0.495) with a relationship of:

$$\frac{\Delta_B}{D} = 0.047 \left( \frac{F_C}{LD} \sqrt{\frac{C_p t}{D^2}} \right)^{0.15}$$  \hspace{1cm} (5.9)
5.5.5 Consolidation Load

The consolidation tests from test series 24H, (64kg consolidation load, five minute consolidation time, 0.006m/s pull-out velocity) were compared with pull-out velocity test series 12 (0kg consolidation load, five minutes consolidation time, 0.006m/s
pull-out velocity). This comparison was intended to show the effect of the consolidation load, and hence the pipe weight on the maximum uplift resistance force and the break out displacement.

A comparison of all of the consolidation tests is shown in Figure 5.15 and a summary of the results is given in Table 5.10. Comparisons are as follows:

- The tests with the higher consolidation load had a higher maximum uplift force, approximately 50% greater for a 500% increase in consolidation load.
- The break out displacement was not observed to change with the increased consolidation load

Table 5.10 – Summary of Consolidation Load Test Data

<table>
<thead>
<tr>
<th>Test</th>
<th>Consolidation Load (kg)</th>
<th>Consolidation Time (mins)</th>
<th>Normalised Consolidation Load and Time (Pa)</th>
<th>Maximum Uplift Force (N)</th>
<th>Normalised Maximum Uplift Force (-)</th>
<th>Normalised Pull-Out Velocity (D/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T12A</td>
<td>15.3</td>
<td>5.0</td>
<td>12.2</td>
<td>170.4</td>
<td>0.37</td>
<td>0.037</td>
</tr>
<tr>
<td>T12B</td>
<td>15.3</td>
<td>5.0</td>
<td>12.2</td>
<td>148.4</td>
<td>0.32</td>
<td>0.037</td>
</tr>
<tr>
<td>T12C</td>
<td>15.3</td>
<td>5.0</td>
<td>12.2</td>
<td>145.7</td>
<td>0.32</td>
<td>0.039</td>
</tr>
<tr>
<td>T12D</td>
<td>15.3</td>
<td>5.0</td>
<td>12.2</td>
<td>155.1</td>
<td>0.34</td>
<td>0.037</td>
</tr>
<tr>
<td>T24H-1</td>
<td>78.9</td>
<td>5.0</td>
<td>63.1</td>
<td>219.6</td>
<td>0.48</td>
<td>0.035</td>
</tr>
<tr>
<td>T24H-2</td>
<td>78.9</td>
<td>5.0</td>
<td>63.1</td>
<td>202.1</td>
<td>0.44</td>
<td>0.037</td>
</tr>
<tr>
<td>T24H-3</td>
<td>78.9</td>
<td>5.0</td>
<td>63.1</td>
<td>220.5</td>
<td>0.48</td>
<td>0.040</td>
</tr>
</tbody>
</table>
Figure 5.15 – Force Vs Displacement Curves with Consolidation Loads

A scatter plot showing the normalised maximum uplift resistance with the normalised consolidation load and time is shown in Figure 5.16. This shows that as the consolidation load increased the maximum uplift resistance force also increased. Unfortunately there were only two sets of data available, hence curve fitting is questionable.

Figure 5.16 – Consolidation Load Vs Maximum Uplift Resistance Force
The data shows that when the consolidation load is increased the corresponding maximum uplift resistance force also increased. This indicates that heavier pipelines and SCRs will be subject to larger uplift resistance forces.

The data from the consolidation load tests was combined with the data from the consolidation time tests. Scatter plots of normalised consolidation load and time with normalised uplift resistance and normalised break-out displacement given in Figure 5.17 and Figure 5.18 respectively. Both trends show that a power law, which was also used with the consolidation time data, gives the best fit for the scatter data for both the maximum normalised uplift resistance force and the normalised break out displacement. These relationships are given below.

\[
\frac{Q_s}{LDS_C} = 0.21 \left( \frac{F_C}{LD} \sqrt{\frac{C_C}{D^2}} \right)^{0.18} \tag{5.10}
\]

\[
\frac{\Delta_B}{D} = 0.042 \left( \frac{F_C}{LD} \sqrt{\frac{C_C}{D^2}} \right)^{0.17} \tag{5.11}
\]

Figure 5.17 – Summary of Both Consolidation Time and Load Vs Uplift Resistance
5.5.6 Pipe Diameter

A series of pull-out tests were conducted using the 0.0484m outer diameter pipe to examine the effect of pipe diameter. The tests used three pull-out velocities, 3mm/s, 6mm/s and 12mm/s which were the same speeds used in the pull-out velocity experiments with the 0.1683m diameter pipe. A summary of the results is given in Table 5.11 with the force/displacement curves shown in Figure 5.19.
Table 5.11 – Summary of Small Diameter Pipe Tests

<table>
<thead>
<tr>
<th>Test</th>
<th>Pull-Out Velocity (mm/s)</th>
<th>Normalised Pull-out Velocity (D/s)</th>
<th>Maximum Uplift Force (N)</th>
<th>Normalised Maximum Uplift Force (-)</th>
<th>Break-out Displacement (m)</th>
<th>Normalised Break-out Displacem’t (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP025</td>
<td>6.3</td>
<td>0.13</td>
<td>35.0</td>
<td>2.3</td>
<td>0.020</td>
<td>0.41</td>
</tr>
<tr>
<td>TP026</td>
<td>6.1</td>
<td>0.13</td>
<td>44.0</td>
<td>2.9</td>
<td>0.020</td>
<td>0.41</td>
</tr>
<tr>
<td>TP027</td>
<td>3.1</td>
<td>0.06</td>
<td>15.5</td>
<td>1.0</td>
<td>0.014</td>
<td>0.30</td>
</tr>
<tr>
<td>TP028</td>
<td>3.1</td>
<td>0.06</td>
<td>28.0</td>
<td>1.9</td>
<td>0.014</td>
<td>0.29</td>
</tr>
<tr>
<td>TP029</td>
<td>3.2</td>
<td>0.07</td>
<td>24.8</td>
<td>1.7</td>
<td>0.013</td>
<td>0.26</td>
</tr>
<tr>
<td>TP030</td>
<td>12.6</td>
<td>0.26</td>
<td>45.6</td>
<td>3.0</td>
<td>0.026</td>
<td>0.54</td>
</tr>
<tr>
<td>TP031</td>
<td>12.2</td>
<td>0.25</td>
<td>46.0</td>
<td>3.1</td>
<td>0.027</td>
<td>0.56</td>
</tr>
</tbody>
</table>

Varying Pull Out Velocities
Unconsolidated Clay, Consolidation Load 12.0kg, Pipe Diameter 0.0484m, Pipe Length 0.948m

Figure 5.19 – Small Diameter Pipe Force/Displacement Curves

Comparisons were made between the small 0.0484m diameter pipe tests and the large 0.1683m diameter pipe tests, Figure 5.20. It can be seen that the relationship between the normalised pull-out velocity and the normalised maximum uplift resistance force was excellent. This indicated that the normalising parameters captured the effect of pull-out velocity and pipe diameter well. The equation of the curve, is given below, and matched the data with a correlation factor of 0.94
\[ \frac{Q_s}{LDS_U} = 7.81\left(\frac{V}{D}\right)^{0.62} \]  

(5.12)

A comparison of normalised break out displacement against normalised pull-out velocity is given in Figure 5.21. This shows that the normalising parameters used did not account for pipe diameter and further assessment was required. A comparison of breakout displacement against normalised pull-out velocity is given in Figure 5.22. This shows that the trend lines for the small and large pipes are coincident, indicating that the break out displacement was proportional to a power of the normalised pull-out velocity. The equation of the curve, is given below, which matched the data which a correlation coefficient of 0.968.

\[ \Delta_B = 0.048\left(\frac{V}{D}\right)^{0.46} \]  

(5.13)

Figure 5.20 – Normalised Uplift Resistance Force Vs Pull-out Velocity
5.5.7 Trench Type

To determine upper bound soil/suction forces and break-out displacements a series of pull-out tests were conducted using a tight trench, as described in Section 5.3.2. The 0.0484m diameter pipe was used with five different pull-out velocities, 3mm/s,
6mm/s, 10mm/s, 13mm/s and 16mm/s. These pull-out velocities were chosen to be comparable with both the STRIDE and CARISIMA (see Appendix D) data sets.

A summary of the pull-out data is given in Table 5.12 and the force/displacement curves in Figure 5.23. The general trends of these tests were consistent with previous experiments; as the pull-out velocity increased so did the uplift resistance force and the breakout displacement.

Table 5.12 – Summary of Pull-out Velocities and Maximum Uplift Force of Pipe in Tight Trench

<table>
<thead>
<tr>
<th>Test</th>
<th>Pull-Out Velocity (mm/s)</th>
<th>Normalised Pull-out Velocity (D/s)</th>
<th>Maximum Uplift Force (N)</th>
<th>Normalised Maximum Uplift Force (-)</th>
<th>Break-out Displacement (m)</th>
<th>Normalised Break-out Displacement (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP013</td>
<td>3.2</td>
<td>0.07</td>
<td>58.3</td>
<td>4.0</td>
<td>0.0232</td>
<td>0.48</td>
</tr>
<tr>
<td>TP014</td>
<td>6.5</td>
<td>0.13</td>
<td>61.3</td>
<td>4.1</td>
<td>0.0274</td>
<td>0.57</td>
</tr>
<tr>
<td>TP015</td>
<td>10.4</td>
<td>0.21</td>
<td>72.9</td>
<td>5.1</td>
<td>0.0318</td>
<td>0.66</td>
</tr>
<tr>
<td>TP016</td>
<td>13.0</td>
<td>0.26</td>
<td>76.4</td>
<td>5.3</td>
<td>0.0354</td>
<td>0.73</td>
</tr>
<tr>
<td>TP022</td>
<td>16.7</td>
<td>0.34</td>
<td>94.4</td>
<td>6.6</td>
<td>0.0388</td>
<td>0.80</td>
</tr>
</tbody>
</table>

Varying Pull Out Velocities in Firm Trench, Consolidated Clay, Consolidation Load 12.0kg, Pipe Diameter 0.0484m, Pipe Length 0.948m

Figure 5.23 – Varying Pull-Out Velocity in Tight Trench
Comparisons between the trends for normalised pull-out velocity and normalised maximum uplift force and break out displacement for both the loose and tight trenches are given in Figure 5.24 and Figure 5.25. These trends show that a power relationship could be used to relate normalised maximum uplift resistance and break out displacement to normalised pull-out velocity. The correlation coefficient for the normalised maximum uplift resistance force is 0.67 indicating a reasonable fit while for the break out displacement it is 0.83, indicating a good fit.

![Figure 5.24 – Pull-out Velocity Vs Uplift Resistance](image)
Normalised Pull-out Velocity Vs Break Out Displacement
Unconsolidated Clay, Pipe Diameters 0.1683m and 0.0484m, Pipe Length 0.948m
Comparison of Loose and Firm Trenches

Figure 5.25 – Pull-out Velocity Vs Break-out Displacement

5.6 Discussion

5.6.1 Comparison of Experimental Results with Theory

Comparisons are conducted between the experimental results and published data by Byrne and Finn (1978). The normalised maximum uplift resistance, pull-out velocity and break out displacement for the repeatability tests are given in Table 5.7. These can be compared to the previous experiments conducted in a triaxial cell by Byrne and Finn (1978), who concluded that the normalised uplift resistance factor, N, was around 6.92 for a pull-out rate of 0.126mm/s. Byrne and Finn (1978) value of N is 6 times higher than reported from the pipe pull-out experiments. The reason for this is due to the difference between the test models used and the failure mechanism of the soil. In the pipe pull-out experiments breakout was observed when the bond between the pipe and the soil failed. The Byrne and Finn experiments used a circular plate surrounded by a skirt. The skirt was driven into the soil and acted in a similar manner to a suction pile. This changed the point of failure from the soil/structure interaction plane to a shear failure within the soil. For this reason the normalised force was observed to be higher in the Byrne and Finn experiments.
The values of the normalised maximum uplift resistance force using the tight trench
were approximately 2.7 times higher than those conducted using the loose trench.
This increase in N was expected as the contact area between the pipe and the soil was
increased. In addition the values for the fast pull-out tests compared well with the
published data by Byrne and Finn (1978) where they reported a minimum value of N
of 6.4 from the pipe/soil suction tests.

5.6.2 Observed Relationships

The experimental data shows that the maximum uplift force and break-out
displacement were both dependant on the pull-out velocity, consolidation time,
consolidation load, pipe diameter and tightness of the trench. The equations to
describe the pipe/soil suction force are given below.

\[
\frac{Q_{S\text{max}}}{LDS_U} = f\left[ k_{FS} \left( \frac{V}{D} \right)^{nF} , k_{TF} \left( \frac{F_c \sqrt{c, t}}{LD^2} \right)^{nTF} \right] \tag{5.14}
\]

\[
\frac{\Delta_B}{D} = f\left[ k_{DS} \left( \frac{V}{D} \right)^{nD} , k_{DTF} \left( \frac{F_c \sqrt{c, t}}{LD^2} \right)^{nDTF} \right] \tag{5.15}
\]

where

\[ k_{FS}, k_{DS}, k_{TF}, k_{DTF}, n_{FS}, n_{DS}, n_{TF} \text{ and } n_{DTF} \]

are empirical constants relating normalised maximum uplift force and normalised break-out displacement to normalised pull-out velocity and normalised consolidation time and load.

The relationships are consistent with published data as the uplift resistance force is
dependant on undrained shear strength, as shown by Vesic (1971) and related to pull-out velocity as shown by Byrne and Finn (1978).

5.6.3 Shape of the Pipe/Soil Suction Curve

Further analysis was conducted to determine the shape of the pipe/soil suction curve.
The experimental data was normalised by dividing the force component of the curve
by the maximum uplift resistance and dividing the displacement component of the
curve by the break-out displacement. This produced a series of modified curves that have a maximum uplift force of 1.0 (dimensionless) and a break-out displacement of
1.0 (dimensionless), Figure 5.26. It can be seen that the modified curves generally have similar shapes and could be simplified into a tri-linear form consisting of:

- an initial linear ramp (suction mobilisation) to the maximum uplift resistance force over 7.5% of the break out displacement,
- the suction plateau which follows from the end of the suction mobilisation and lasts until 30% of the break out displacement, and
- the suction release which follows from the end of the suction plateau and linearly ramps to zero force at the break out displacement.

![Varying Pull Out Velocities](image)

Figure 5.26 – Normalised Experimental Data with Tri-linear Soil Curve Model

### 5.7 Summary and Conclusions

The STRIDE 2D pipe/soil interaction tests showed that when a pipe was pulled vertically upwards from a trench in a saturated subsea clay an uplift resistance force was mobilised. This force was due to the adhesion of the clay to the pipe. The tests conducted showed that the magnitude of the uplift resistance force and the break out displacement were proportional to pull-out rate, consolidation time, consolidation load (pipe weight), pipe diameter and the tightness of the trench.
5.7.1 Relationships

The trends from the experimental data showed that the normalised maximum uplift resistance was related to a power of the normalised pull-out velocity and a power of the normalised consolidation time and load. These equations also took into account the effect of the riser diameter, but did not account for trench depth as all experiments were conducted at the same trench depth. The break out displacement was also related to a power of normalised pull-out velocity and a power of the consolidation time and load. It was also shown that the normalised break out displacement parameter did not depend on pipe diameter.

The normalised maximum uplift resistance parameter used in these experiments was derived from bearing capacity formula, and equates to N. The other normalising parameters were assumed to modify the N value, increasing it if the pull-out velocity or consolidation time/load increases. It was assumed that the trench tightness can be accounted for in this way by using the solution for pipe penetration with varying soil coverage as given by Murff et al (1989). This method, while not used here, was used in subsequent sections that detail the development of the pipe/soil interaction models.

5.7.2 Comparison to a SCR

The STRIDE 2D small scale tests were designed to represent a small section of a SCR in the TDZ. The pullout tests for a pipe diameter were all conducted in the same trench. This was done to represent the actual conditions of the TDZ where a riser would cycle in the same trench. In addition the pipe was pulled using wires connected to both ends of the pipe, which allowed one end to 'pop' up before the other, similar to the 3D pipe peeling effect observed within the full scale tests. A test conducted with a rigid pipe, unable to peel away from the soil would have provided a conservative upper bound force/displacement curve.

Many of the environmental forces acting on the full scale pipe were removed from small scale tests, such as current scouring around the pipe, indicating that only the pipe soil interaction will be examined. No material transport or trenching mechanisms, except pipe penetration, consolidation and settlement will occur.
5.7.3 Assessment of Tests

The STRIDE 2D small scale tests were designed to compliment the earlier STRIDE full scale tests which also examined pipe/soil interaction. Most of the pull-out velocities and consolidation times used were chosen to match the data from these tests. The objectives of the small scale tests were met; to obtain a data set from which a comprehensive pipe/soil suction model could be developed and used in SCR analysis.

The tests were conducted on a relatively small budget that required simplification in the experimental apparatus to be made. Ideally an automatic computer controlled actuator would have been used, but this was not available. However the hand actuation of the pull-out tests was shown to be repeatable and give consistent results. The load cell and the displacement sensor measured the load and pipe movement accurately, within their given tolerances.

The parameters examined cover a broad range of the expected motions in a full scale SCR. The short consolidation time using the remoulded shear strength represents pipe/soil interaction during day-to-day motions. The longer consolidation times were more representative of second order and slow drift motions. It would have been beneficial to have had a very long consolidation test, where the pipe was allowed to consolidate over a period of months. This would show the effect on a failed mooring line event where the vessel moves a large distance, pulling with it a section of the pipe that would normally rest on the seabed. However it was thought that this scenario could be interpolated from the existing data taking into account settlement and consolidation theory for strip foundations resting on a clay soil.
6.0 A MODEL FOR PIPE/SOIL SUCTION

6.1 Introduction

The previous chapter has detailed the STRIDE (2H Offshore, 2002a) 2D pipe pull-out experiments that were conducted by the writer to examine the effect of pipe/soil suction between a pipe and a clay soil. This section uses the results and relationships observed in the STRIDE tests, together with the results from a second data set examining pipe pull-out from clay soils from the CARISIMA JIP (Marintek, 2000a) and (Marintek, 2000b) to construct a vertical upwards pipe/soil interaction model. Details of the CARISIMA tests are given in Appendix D with an overview of the testing programme below.

Phase I of the CARISIMA JIP was conducted at the same time as the STRIDE 2D pipe/soil interaction tests were being conducted. The main objectives of the CARISIMA JIP was to examine the effect of pipe/soil suction on a pipe being pulled vertically upwards out of a trench in order to create a pipe/soil suction model that could be programmed into the Riflex finite element analysis code (Marintek, 1997). As part of a data exchange between STRIDE and CARISIMA, STRIDE was able to obtain the results from all of the vertical pull-out tests conducted by CARISIMA on the Onsoy clay, a marine sediment with similar properties to deepwater Gulf of Mexico clays (Marintek, 2000a), however the relationships and pipe/soil suction model derived by CARISIMA remained confidential. Later it was agreed that STRIDE would supply CARISIMA with enough Watchet Harbour clay to conduct a series of pipe/soil interaction tests using the CARISIMA test rig. Again only the test results were supplied to STRIDE.

6.2 Correlating STRIDE and CARISIMA Tests

6.2.1 Comparison of Test Methods

The STRIDE and CARISIMA JIPs used different test rigs and procedures to conduct the pipe pull-out experiments, and examine and quantify the pipe/soil suction force. An initial comparison is conducted on the diameters and lengths of the STRIDE and CARISIMA test pipes. A comparison of the differences between the test pipes is given in Figure 6.1. This shows that both the STRIDE and CARISIMA test pipes had
aspect ratios greater than 4, which indicates that end effects in the tests should be negligible.

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Diameter x Depth</th>
<th>Aspect Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>STRIDE 1</td>
<td>0.0484m x 0.948m</td>
<td>19.6</td>
</tr>
<tr>
<td>STRIDE 2</td>
<td>0.1683m x 0.948m</td>
<td>5.6</td>
</tr>
<tr>
<td>CARISIMA 1</td>
<td>0.1016m x 0.4064m</td>
<td>4.0</td>
</tr>
<tr>
<td>CARISIMA 2</td>
<td>0.2191m x 0.8764m</td>
<td>4.0</td>
</tr>
</tbody>
</table>

Figure 6.1 – Comparison of the STRIDE and CARISIMA Test Pipes

The test rig used in the CARISIMA experiments consisted of a pipe that was rigidly connected to a H-beam that could be precisely raised or lowered by a computer controlled servo-hydraulic actuator into the clay soil. In addition the rig could actuate the pipe horizontally, allowing lateral tests as discussed in subsequent chapters. The CARISIMA tests were conducted in a firm trench that was created by penetrating the pipe into a virgin soil, and as such was assumed to represent the upper bound solution. Every test was conducted on virgin soil. Due to cost restraints the number of tests that could be completed was limited. The CARISIMA tests also examined a number of other possible influences on the pipe/soil suction force including venting (where the soil at the end of the pipe is removed to allow water to flow into and disrupt the bond between the soil and the pipe), remoulding (where the soil is intentionally stirred up prior to testing to produce a remoulded, non-consolidated soil) and pre-cycling (where the pipe is cycled in the trench prior to consolidation or pull-out or both).

The STRIDE test used strops attached to the ends of the pipe that allowed only vertical pull-out tests. The strops allowed the pipe to peel away from the trench and
were assumed to represent the actual SCR pipe/trench interaction. In addition each pull-out test was conducted in an existing trench, again representative of the actual SCR touchdown point motions. Since each test was not conducted on a virgin soil there was no limit to the number of pull-out tests that could have been conducted; most tests were repeated at least three times with consistent results. A summary of the differences between the STRIDE and CARISIMA test rigs is given in Table 6.1.

**Table 6.1 – Comparison of Test Methods**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>STRIDE</th>
<th>CARISIMA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actuation Method</td>
<td>Hand crank timed to metronome</td>
<td>Computer controlled servo-hydraulic pistons</td>
</tr>
<tr>
<td>Clay Preparation</td>
<td>All tests conducted in existing trenches to match SCR conditions</td>
<td>All tests conducted on virgin clay</td>
</tr>
<tr>
<td>Pipe Lift</td>
<td>Pipe allowed to peel away from trench, same mechanism observed in SCRs</td>
<td>Pipe was pulled vertically upwards out of trench in a flat lift</td>
</tr>
<tr>
<td>Trench Type</td>
<td>Either firm of loose trench</td>
<td>Firm trenches only</td>
</tr>
<tr>
<td>Number of Tests</td>
<td>Many tests conducted with each combination of parameters repeated at least 3 times</td>
<td>All tests conducted on virgin seabed, indicating a limited number of tests</td>
</tr>
<tr>
<td>Measured Data</td>
<td>Forces and displacements</td>
<td>Forces, displacement, accelerations and pressure sensors on bottom of pipe</td>
</tr>
</tbody>
</table>

From the observations of the differences of the testing programs it was concluded that the CARISIMA data provided accurately measured and potentially conservative test data, due to the rigid pipe lift. The STRIDE 2D tests provided a large number of repeated tests at a range of values for each of the parameters tested, and therefore were used primarily to determine the relationships between the parameters. These relationships could then be confirmed with the CARISIMA data.

### 6.2.2 Comparison of Force Displacement Curves

A set of tests was conducted using the STRIDE test rig to match some of the tests conducted in CARISIMA. The pipe diameter and length were the main parameters that were different, while the consolidation time and pull-out velocity were matched
as closely as possible. All tests were conducted in firm trenches at an embedment depth of 0.5D. A summary of the test parameters is given in Table 6.2 and a comparison of the geotechnical parameters between the Onsoy and Watchet Harbour clay in Table 6.3. The test results were compared directly using the normalisation parameters described previously.

Table 6.2 – Comparison of Test Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>STRIDE</th>
<th>CARISIMA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe Diameter</td>
<td>0.0484m</td>
<td>0.1016m</td>
</tr>
<tr>
<td>Pipe Length</td>
<td>0.948m</td>
<td>0.4064m</td>
</tr>
<tr>
<td>Aspect Ratio</td>
<td>19.6</td>
<td>4.0</td>
</tr>
<tr>
<td>Consolidation Time</td>
<td>5.5, 16 and 26 hours</td>
<td>17 to 20 hours CARISIMA I</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2 hours CARISIMA II</td>
</tr>
<tr>
<td>Pipe Mass</td>
<td>12.0kg</td>
<td>-</td>
</tr>
<tr>
<td>Pre-load</td>
<td>0.0N</td>
<td>100N</td>
</tr>
<tr>
<td>Pull-out Velocity</td>
<td>10.0mm/s</td>
<td>10.0mm/s</td>
</tr>
<tr>
<td>Trench Type</td>
<td>Firm</td>
<td>Firm</td>
</tr>
<tr>
<td>Pipe Lift</td>
<td>Pipe Peeling</td>
<td>Flat Lift</td>
</tr>
</tbody>
</table>

Table 6.3 – Comparison of Geotechnical Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Watchet Harbour Clay</th>
<th>Onsoy Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plasticity Index (%)</td>
<td>50</td>
<td>30</td>
</tr>
<tr>
<td>Average Soil Strength at ½D (kPa)</td>
<td>1.1*, 3.0‡</td>
<td>2.5*</td>
</tr>
<tr>
<td>Coefficient of Consolidation (m²/year)</td>
<td>0.2</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Notes
* Undrained shear strength at ½D for the 0.0484m Do pipe
‡ Undrained shear strength at ½D for the 0.1016m Do pipe
Comparisons of the force/displacement curves from the pull-out tests are shown in Figure 6.2 and non-dimensional curves in Figure 6.3. A summary of the results is given in Table 6.5. Observations from the comparisons of the STRIDE and CARISIMA tests follow.

The force/displacement curves from the STRIDE and CARISIMA tests had similar trends, both sets of curves started at the origin, increased to a maximum force, plateau, then reduced to zero. The STRIDE tests were observed to have the same length suction release (the difference between the break-out displacement and the plateau distance) as the CARISIMA phase I tests, which was longer than the suction release on the CARISIMA phase II tests that were conducted on the same clay.

Analysis was conducted to compare the shape of the STRIDE and CARISIMA pipe/soil suction force/displacement curves by the method described in section 5.6. The results of the analysis are given in Table 6.4 and show that the distance of the suction release was approximately 70% of the STRIDE curves while only 30% of the CARISIMA curves. This effect is thought to be due to the pipe peeling away from the trench, where one end of the pipe breaks-out while the other end continues to experience the pipe/soil suction force. This was allowed during the STRIDE tests but not allowed during the CARISIMA tests.

<table>
<thead>
<tr>
<th>Description</th>
<th>% of Plateau Displacement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CARISIMA</td>
</tr>
<tr>
<td>Mobilisation Distance</td>
<td>7.5</td>
</tr>
<tr>
<td>Plateau Distance</td>
<td>70</td>
</tr>
<tr>
<td>Break-out Displacement</td>
<td>100</td>
</tr>
</tbody>
</table>

The maximum pull-out forces in the CARISIMA tests were observed to be between 350N and 500N, approximately 90% to 180% higher than in the STRIDE tests. This difference was expected, as the pipe diameter of the STRIDE tests was under half that of the CARISIMA tests. Comparing the non-dimensional curves in Figure 6.3 showed that the maximum non-dimensional pipe/soil suction forces were around 3.0
for the CARISIMA II tests and the STRIDE test with the low consolidation time. This indicated that the maximum non-dimensional forces from the STRIDE tests were comparable to those of similar CARISIMA tests. The remaining STRIDE tests had a maximum non-dimensional force of 3.5 that was lower than the CARISIMA I tests where the maximum non-dimensional pipe/soil suction force was 4.5. This difference was due to the different clays used in the tests and the pipe pull-out method, the CARISIMA tests were expected to yield higher maximum forces than the equivalent STRIDE tests. The non-dimensional break-out displacements for the STRIDE, CARISIMA I and CARISIMA II tests were around 0.7, 0.33 and 0.25 respectively. This shows that the STRIDE breakout displacement was approximately three times that of the equivalent CARISIMA tests.

Table 6.5 – Summary of Comparable STRIDE and CARISIMA Test Data

<table>
<thead>
<tr>
<th>Test</th>
<th>Consolidat’n Time (hours)</th>
<th>Pull-out Velocity (mm/s)</th>
<th>Maximum Uplift Resistance (N)</th>
<th>Maximum Uplift Resistance (Q/[LDSu])</th>
<th>Break-out Displace’nt (m)</th>
<th>Break-out Displace’nt (m/D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>STRIDE TP018</td>
<td>16.0</td>
<td>10.0</td>
<td>180.1</td>
<td>3.65</td>
<td>0.0318</td>
<td>0.657</td>
</tr>
<tr>
<td>STRIDE TP019</td>
<td>26.0</td>
<td>10.0</td>
<td>173.1</td>
<td>3.51</td>
<td>0.0357</td>
<td>0.738</td>
</tr>
<tr>
<td>STRIDE TP020</td>
<td>5.5</td>
<td>10.0</td>
<td>151.1</td>
<td>3.00</td>
<td>0.0315</td>
<td>0.650</td>
</tr>
<tr>
<td>CARISIMA I, Test 1</td>
<td>20.2</td>
<td>10.0</td>
<td>364.2</td>
<td>4.30</td>
<td>0.0335</td>
<td>0.330</td>
</tr>
<tr>
<td>CARISIMA I, Test 3</td>
<td>17.5</td>
<td>10.0</td>
<td>465.1</td>
<td>4.49</td>
<td>0.0335</td>
<td>0.330</td>
</tr>
<tr>
<td>CARISIMA I, Test 11</td>
<td>18.0</td>
<td>10.0</td>
<td>386.4</td>
<td>4.54</td>
<td>0.0335</td>
<td>0.330</td>
</tr>
<tr>
<td>CARISIMA II, Test 1-1</td>
<td>2.0</td>
<td>10.0</td>
<td>501.8</td>
<td>3.20</td>
<td>0.0248</td>
<td>0.244</td>
</tr>
<tr>
<td>CARISIMA II, Test 4-1</td>
<td>10min</td>
<td>10.0</td>
<td>472.4</td>
<td>2.90</td>
<td>0.0240</td>
<td>0.236</td>
</tr>
</tbody>
</table>
6.2.3 Observation of Trends in the CARISIMA Tests

Comparisons were made between the firm trench results from the STRIDE tests with the CARISIMA pull-out tests. These were conducted to confirm the trends observed
in the STRIDE data with the CARISIMA data. The first comparison was between the maximum pull-out force and the pull-out velocity, Figure 6.4. This showed that, as in the STRIDE data, the maximum pull-out force increased with pull-out velocity. This plot also showed that the relationship of the trend lines of the STRIDE, CARISIMA I and CARISIMA II – 1.5D trench depth tests was similar; the power was between 0.18 and 0.21 and the constant was between 6.4 and 7.0. The trend of the tests from CARISIMA II – 0.5D trench depth was similar to the others as the maximum pull-out force increased with pull-out velocity and a power relationship provided a good fit to the data, however it was observed that the maximum forces were approximately 25% lower than the other trends. This lower force was due to the low consolidation time of ten minutes used in the CARISIMA II – 0.5D trench depth tests, as opposed to around 16 hours for the other CARISIMA tests. This was consistent with the results observed in the previous section, and with the comparisons shown later in this section.

Comparisons were conducted between the STRIDE and CARISIMA data to examine the differences in the trends between break out displacement and pull-out velocity. Initial comparisons used the normalised break out displacement and normalised
pull-out velocity as shown in Figure 6.5 and showed that in both STRIDE and CARISIMA data sets the break out displacement increases with increasing pull-out velocity and that the gradient were approximately equal. It was also observed in the CARISIMA data that the normalising method used does not fully account for pipe diameter, otherwise the blue and green lines (0.1016m and 0.2191m diameter pipe, respectively) would coincide. This conclusion was the same as observed in the previous section for the STRIDE data. A series of comparisons were conducted and it was observed that the lines coincide when normalised breakout displacement was plotted with pull-out velocity, Figure 6.6. This result was different to the STRIDE tests where the best relationship was provided using break out displacement and normalised pull-out velocity.

Figure 6.5 – Normalised Break-out Displacement Vs Normalised Pull-Out Velocity
The effects of consolidation load and consolidation time were assessed within the CARISIMA II JIP. The data showed that as the consolidation time and load increased the uplift force also increased, Figure 6.7. The comparison with the STRIDE data shows large discrepancies because the STRIDE tests used a lower pull-out velocity and were conducted in a loose trench. The general trend from the break-out displacement, Figure 6.8, was that the break out displacement increased with increasing consolidation time and load factor and that a power relationship gives the best fit.
6.3 Observations from the CARISIMA Tests

The CARISIMA tests examined the effect of venting on the pull-out experiments, where the pipe was pushed into the clay, and prior to testing a section of clay at one
end of the pipe was removed. This allowed water to flow into the gap between the pipe and the clay as the pipe was pulled out, and was intended to show the peeling effect. The resulting force/displacement curves are given in Figure 6.9 and show that venting the pipe (test V13) reduces the maximum pull-out force by approximately 10% and the break-out displacement by approximately 16%. This indicated that the effect of water flowing between the pipe and the soil as the pipe was pulled out did not have a great effect on the pipe/soil suction bond.

The effect of conducting a pullout test on remoulded clay is shown in Figure 6.9. This shows that the normalised uplift resistance force and the break-out displacement were similar when the undrained soil shear strength was modified accordingly (i.e. the remoulded test used the remoulded undrained shear strength). This indicated that the pull-out force was dependent on the undrained shear strength.

The effect of cycling on the pull-out force is shown in Figure 6.9. If the pre-cycling (moving the pipe vertically in the trench for approximately 100 cycles between zero and 100N) was conducted prior to the consolidation period (test V7) then the effect on the force/displacement curve was negligible. If the pre-cycling was conducted after consolidation and prior to pull-out then the maximum pull-out force reduced by approximately 74% while the break-out displacement reduced by 16%, to the same value as in the vented experiment. This indicated that if the pipe or riser was consistently cycling, the pull-out force experienced was low.
Further tests were conducted to examine the effect of repeated cycling with break out (i.e. during each pull-out and repenetration cycle the pipe loses contact with the soil) on the pull-out force. The 0.1016m outer diameter pipe was pushed into the trench, left to consolidate then pulled out (as previous tests). After the first pull-out the pipe was pushed back into the trench, left to consolidate for five minutes and a subsequent pull-out test conducted. This was repeated five times. The results of this experiment are summarised in Figure 6.10 and show that the first pull-out of the cycling tests had a maximum force of approximately 95N. The second pull-out had a force of 71N, 75% of the first pull. The subsequent tests had maximum pull-out forces between 49N and 54N that was approximately 56% of the force of the first pull-out test. This showed that repeated cycling of a pipe on the soil disturbed the pipe/soil suction bond and reduced the uplift resistance force experienced by the pipe. The effect of cycling the pipe in the trench was also observed on the break-out displacement, where the break-out displacement reduced from approximately 0.092m to 0.0083m, a reduction of approximately 10%.
6.4 Development of Pipe/Soil Suction Model

Using the experiments conducted within the STRIDE and CARISIMA JIPs and the observations detailed in this and the previous section a pipe/soil suction model was developed for use in finite element analysis codes. The pipe/soil suction model had to account for both static and dynamic analysis.

6.4.1 Shape of the Pipe/Soil Suction Model

The pipe/soil suction model was based on experimental data described previously. For analysis purposes this was modelled as a piece-wise linear curve in three linear phases as shown in Figure 6.11 and described below.

- Suction mobilisation – As the pipe initially moved upwards the suction force increased from zero to the maximum value
- Suction plateau – The suction force remained constant as the pipe moved further upwards
- Suction release – Under further upward movement the suction force reduced from its maximum to zero at the break-out displacement
The pipe/soil suction model had two defined limits, the maximum uplift resistance force and the break-out displacement from which all points on the pipe/soil suction model were derived. The values used for each of these parameters depended on the type of SCR analysis conducted.

![Graph depicting uplift resistance force and suction mobilisation](image)

**Figure 6.11 – Observed Pipe/Soil Curve and Pipe/Soil Curve Used in Analysis**

### 6.4.2 Maximum Pipe/Soil Suction Force

The maximum pipe/soil suction force was determined based on the normalised force equation and strip foundation theory bearing capacity formulas. In addition to the pipe length, pipe diameter and the undrained shear strength it was observed that the pull-out force was dependant on hysteresis (cycling prior to pull-out), pull-out velocity, consolidation time and consolidation load. This was described using the formula below.

\[
Q_{s, \text{MAX}} = k_c \times k_y \times k_T \times N \times L \times D \times S_U
\]

where

- \(Q_{s, \text{MAX}}\) is the maximum pipe/soil suction force
- \(N\) is the non-dimensional shape and depth factor
- \(L\) is the length of pipe
D is the diameter of pipe
$S_U$ is the undrained shear strength of clay
$k_C$ is the cyclic loading factor (dimensionless)
$k_V$ an empirical pull-out velocity factor (dimensionless)
$k_r$ is the consolidation time factor (dimensionless)

From the comparisons of the STRIDE and CARISIMA experimental data the empirical pull-out velocity factor was estimated using a power relationship to the normalised velocity as shown below.

$$k_V = k_F \left( \frac{V}{D} \right)^{n_F} \quad (6.2)$$

where

- $V$ is the vertical pull-out velocity
- $k_F$ is an empirically derived constant relating the dimensionless pull-out velocity factor to the normalised pull-out velocity to a power ($s^{n_F}$)
- $n_F$ is an empirically derived constant for the power of normalised pull-out velocity (dimensionless)

The empirical consolidation load and time factor was estimated using a power relationship to the normalised load and time as shown below.

$$k_r = k_{TF} \left( \frac{F_C \sqrt{c_v t}}{L D^2} \right)^{n_{TF}} \quad (6.3)$$

where

- $c_v$ is the coefficient of consolidation
- $t$ is the consolidation time
- $F_C$ is the consolidation load
- $k_{TF}$ is an empirically derived constant relating the consolidation load and time factor to a power of the normalised consolidation load and time ($m^2/N)^{n_{TF}}$
- $n_{TF}$ is an empirically derived constant for the power of the consolidation load and time (dimensionless)
The empirical factors used in the models were based on the test data. The first empirical parameters derived were for the pull-out velocity. This was conducted by examining the trend of the normalised pull-out velocity with the normalised maximum pull-out force divided by $N$. Since the relationship was a power relationship dividing the normalised force by $N$ has the same effect as dividing the empirical factor $k_F$ by $N$. The values for the Onsoy clay were calculated using those given previously, while the values for the Watchet clay were the best fit trend of both the STRIDE and CARISIMA data as show in Figure 6.12. The results for $k_F$ and $n_F$ are summarised in Table 6.6 for the two clays. For completeness the plasticity index of the clays is also given.

**Table 6.6 - Empirical Factors for Maximum Uplift Force**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Clay Type</th>
<th>Onsoy</th>
<th>Watchet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plasticity Index (%)</td>
<td>30</td>
<td>42</td>
<td></td>
</tr>
<tr>
<td>$k_F$</td>
<td>1.11</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td>$n_F$</td>
<td>0.17</td>
<td>0.22</td>
<td></td>
</tr>
</tbody>
</table>

Normalised Maximum Uplift Resistance / Bearing Capacity Factor, $N$ with Normalised Pull Out Velocity

**Figure 6.12 – Determining Empirical Factors for Pull-out Velocity**
The empirical factors for consolidation time and load were determined by dividing the force and consolidation relationship through by \( N \) and \( k_v \), which was calculated using the empirical constants previously determined. The factors were developed for both loose and firm trenches using the data from the Watchet Harbour clay and summarised in Table 6.7.

Table 6.7 - Empirical Factors for Maximum Uplift Force

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Clay Type</th>
<th>Watchet (STRIDE Loose Trench)</th>
<th>Watchet (CARISIMA Firm Trench)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( k_{TF} )</td>
<td>( 0.07 )</td>
<td>( 0.52 )</td>
<td></td>
</tr>
<tr>
<td>( n_{TF} )</td>
<td>( 0.18 )</td>
<td>( 0.10 )</td>
<td></td>
</tr>
</tbody>
</table>

The undrained shear strength used in the model was the shear strength at the bottom of the pipe. When the model was used in dynamic analysis where the loading, for example, was from extreme storm and first-order fatigue and the consolidation time between pipe/soil interaction cycles was short, then the remoulded undrained shear strength was used with the consolidation time factor set to 1.0. For long consolidation times (i.e. greater than five minutes) equation (5.3) was used with the undisturbed undrained shear strength. This is summarised in Table 6.8.

The cyclic loading factor was based upon the observations from the STRIDE and CARISIMA tests and accounts for hysteresis and repeated cycling effects. If the model was used to calculate the pipe/soil suction force on the first pull this factor was set to 1.0. If the model was used in a dynamic analysis where the pipe/soil suction force was subject to cycling then from Figure 6.10 the value of \( k_c \) was suggested to be 0.56 and the soil assumed to be remoulded. A summary of these factors is given in Table 6.6. The effect of pipe peeling was assumed to be negligible.
A comparison between the test data and the model was conducted. The maximum pull-out force from the tests was compared to the force calculated using the model as shown in Figure 6.13. This showed that overall the model gave a good estimate of the pipe/soil suction force as the correlation factor for the comparison was 0.94. The gradient of the trend line was 0.863 and indicates that the model under predicts the pipe/soil suction force. Close inspection showed that the CARISIMA data was predicted well, and the error that causes the under prediction is from the STRIDE data. This was acceptable since there was a large amount of scatter in the STRIDE test data that could be ±20% and the error from the model was within this limit.

![Comparison Between Real and Calculated Data](image)

**Figure 6.13 – Comparison of Test Data and Model**
6.4.3 Break-out Displacement

The break-out displacement was developed using the relationships found earlier in this and previous sections. The model was based on the observations that the break-out displacement was proportional to pipe diameter, pull-out velocity and consolidation time and load. It was also observed that the mobilisation and plateau distances were proportional to the break-out displacement. These observations were represented using the following formula.

\[ \Delta_B = k_{DC} \times k_{DV} \times k_{DT} \times D \]  (6.4)

where

- \( \Delta_B \) is the break-out displacement,
- \( k_{DC} \) is the cyclic loading factor (dimensionless),
- \( k_{DV} \) is the pull-out velocity factor (dimensionless),
- \( k_{DT} \) is the consolidation time factor (dimensionless).

The cyclic loading factor represents the degrading of the pipe/soil suction bond with pipe cycling in the trench. From the test data it was observed that in dynamic analysis this should be taken as 0.9. To model the first pull-out of a pipe in a trench the value of \( k_{DC} \) should be 1.0. For conservatism, the effect of pipe peeling was assumed to be negligible.

The empirical pull-out velocity factor was estimated using a power of the pull-out velocity as shown below:

\[ k_{DV} = k_D \times V^{n_D} \]  (6.5)

where

- \( k_D \) is an empirically derived constant relating the pull-out velocity factor to the pull-out velocity \( ([\text{m/s}]^{n_D}) \)
- \( n_D \) is an empirically derived constant for the power of pull-out velocity (dimensionless)

The empirical consolidation time and load factor was estimated using a power law relationship to the normalised consolidation load and time as shown below:
\[ k_{DT} = k_{DTF} \left( \frac{F_C \sqrt{c_v t}}{L D^2} \right)^{n_{DTF}} \]  

(6.6)

where

- \( k_{DTF} \) empirically derived constant relating the consolidation time and load factor to a power of the normalised consolidation time and load (m²/N)
- \( n_{DTF} \) empirically derived constant for the power of the normalised consolidation time and load (dimensionless)

The empirical factors used in the models were based on the STRIDE and CARISIMA test data. The constants used to calculate the empirical pull-out velocity factor were based on the relationships found in Figure 6.6. The empirical factors derived were different for the two clays and are shown in Table 6.9 below.

**Table 6.9 – Empirical Factors for Break Out Displacement**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Clay Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Onsoy</td>
</tr>
<tr>
<td>Plasticity Index (%)</td>
<td>30</td>
</tr>
<tr>
<td>( k_D )</td>
<td>0.98</td>
</tr>
<tr>
<td>( n_D )</td>
<td>0.26</td>
</tr>
</tbody>
</table>

The empirical factors for the consolidation time and load were derived from the relationships observed in Figure 6.8. The constants observed were normalised with the average non-dimensional break-out displacements since the empirical pull-out velocity factors include a break-out displacement/diameter scaling factor. The derived empirical constants for the Onsoy and Watchet harbour clays are given in Table 6.10 below.
Table 6.10 – Empirical Factors for Break Out Displacement

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Clay Type</th>
<th>Onsoy</th>
<th>Watchet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plasticity Index (%)</td>
<td>30</td>
<td>42</td>
<td></td>
</tr>
<tr>
<td>$k_{DIF}$</td>
<td>0.6</td>
<td>0.4</td>
<td></td>
</tr>
<tr>
<td>$n_{DIF}$</td>
<td>0.1</td>
<td>0.17</td>
<td></td>
</tr>
</tbody>
</table>

A comparison between the break-out displacement from the test data and those calculated using the model was conducted and the results are shown in Figure 6.14. The figure shows that the break-out displacement model gave a good estimate of the break-out displacement as the correlation coefficient was 0.85.

![Comparison Between Real and Calculated Data](image)

Figure 6.14 – Comparison of Real and Calculated Break out Displacements
6.4.4 Limits of Model

The empirical parameters used in the pipe/soil suction models were based upon the STRIDE and CARISIMA experimental work, and therefore were applicable for the ranges of the parameters considered in the tests. A summary of the ranges of the experimental parameters, and therefore the pipe/soil suction model limits are given in Table 4.

Table 6.11 – Limitations of the Pipe/soil Interaction Model

<table>
<thead>
<tr>
<th>Test Parameter</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pull-out Velocity</td>
<td>0.005m/s (0.005V/D)</td>
<td>0.2m/s (0.8V/D)</td>
</tr>
<tr>
<td>Consolidation Time</td>
<td>5min</td>
<td>112Hrs</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>30%</td>
<td>50%</td>
</tr>
</tbody>
</table>

6.4.5 Comparison with Coarse Harbour Test Riser Model

A comparison between the coarse pipe/soil suction model used in the Harbour Test Riser analysis in Chapter 4 and the pipe/soil suction model created using the equations given above. The values used for pull-out velocity, consolidation time and load and the empirical factors are given in Table 4.12. The pipe/soil suction curves are given in Figure 4.37. These show that the maximum pull-out forces between the coarse harbour test model and the pipe/soil suction model were similar, differing by less than 1.5% while the break out displacements differed by only 13%. The main difference was in the shape of the three phase curve, where the mobilisation distance in the coarse harbour test riser model is 0.051m, 42% of the break out displacement compared to 7.5% of the break out displacement from the pipe/soil suction model.
Table 6.12 – Pipe/Soil Suction Model Properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Harbour Test Riser Model</th>
<th>Pipe/Soil Suction Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pull-out Velocity</td>
<td>0.1 m/s</td>
<td>0.1 m/s</td>
</tr>
<tr>
<td>Empirical Pull-out Velocity Factor, k_v</td>
<td>-</td>
<td>0.927</td>
</tr>
<tr>
<td>Consolidation Time, t</td>
<td>16 hours</td>
<td>16 hours</td>
</tr>
<tr>
<td>Consolidation Load, F_C</td>
<td>64 kg</td>
<td>64 kg</td>
</tr>
<tr>
<td>Empirical Consolidation Factor, k_T</td>
<td>-</td>
<td>1.0</td>
</tr>
<tr>
<td>Cyclic Loading Factor, k_C</td>
<td>-</td>
<td>1.0</td>
</tr>
<tr>
<td>Maximum Pipe/soil Suction Force</td>
<td>812 N/m</td>
<td>823 N/m</td>
</tr>
<tr>
<td>Break-out Displacement</td>
<td>0.122 m</td>
<td>0.107 m</td>
</tr>
<tr>
<td>Mobilisation Distance</td>
<td>0.051 m (42% Δ_B)</td>
<td>0.008 m (7.5% Δ_B)</td>
</tr>
<tr>
<td>Plateau Distance</td>
<td>0.089 m (73% Δ_B)</td>
<td>0.075 m (70% Δ_B)</td>
</tr>
</tbody>
</table>

Comparison of 2D Pipe/Soil Suction Curves
From STRIDE II 2D Pipe/Soil Interaction Tests on Watchet Harbour Clay, Willis (2001)
And Pipe/Soil Interaction Model

Figure 6.15 – Comparison of Pipe/Soil Suction Models
6.4.6 Comparison with Published Models

The pipe/soil suction force model was similar to the models developed by Muga (1968), Vesic (1971) and Foda (1983). The force was shown to be proportional to the undrained shear strength and the pull-out velocity. The additional parameters derived in this pipe/soil suction model account for consolidation load, consolidation time and hysteresis as well as developing a model for the break-out displacement, which has not been observed in the available literature. The pipe/soil interaction model has been designed for use in numerical analysis where a pipe/soil interaction curve needs to be defined. This has also not been observed in the available literature.

6.5 Summary and Conclusions

6.5.1 Overview

A numerical model for determining the pipe/soil suction force and the shape of the pipe/soil suction curve has been developed. The model uses empirical factors to relate pull-out velocity, consolidation time, penetration depth and pipe weight to the pipe/soil suction force and the break-out displacement based on STRIDE and CARISIMA pipe/soil interaction data. The model developed was unique in that it was the first model to describe a pipe/soil interaction curve for use in numerical analysis.

6.5.2 STRIDE and CARISIMA Data Sets

The STRIDE and CARISIMA data sets complement each other, providing pull-out data from two different sediments, Watchet Harbour and Onsoy Clay respectively. The data sets complement each other well, the accurate and conservative but limited in number CARISIMA tests and the large range of parameters and great in number STRIDE tests. It was unfortunate that the tests were only conducted on these two clays as any trend derived between the empirical factors and the plasticity index will be linear. Experiments conducted on more data sets would enable better understanding of the effect of the plasticity index on pipe/soil suction.
6.5.3 Development of Pipe/Soil Suction Model

The pipe/soil suction model relates the undrained shear strength and plasticity index soil parameters and the trench depth, pull-out velocity, consolidation time, consolidation load, hysteresis and cyclic loading to pull-out force and break-out displacement. The pipe/soil interaction model had a shape that is defined as a three phase non-linear curve that was bounded by the maximum pull-out force and the break-out displacement. The pipe/soil interaction model provided a framework for implementing pipe/soil suction forces into finite element analysis codes.

Comparisons between the pipe/soil interaction model and the test data showed that the pipe/soil interaction model calculated the pipe/soil interaction curve with good accuracy. The correlation coefficients between the calculated and measured test data were 0.97 and 0.85 for the maximum pull-out force and break-out displacement presciently. Comparisons between the pipe/soil suction curve developed during the harbour test riser and the pipe/soil suction model showed that the maximum pipe/soil suction force was calculated to within 1.5% and the break out displacement to within 13%. Another refinement was the shape of the pipe/soil suction model which had a steeper suction mobilisation and longer suction plateau.

The limitations of the model were determined from the ranges of the STRIDE and CARISIMA test parameters. For pull-out velocity the model was applicable between 0.005m/s and 0.2m/s that covers most SCR motions and consolidation times between five minutes and 112 hours. However longer consolidation times could be conservatively estimated using the equations developed. The range of plasticity index was 30% to 50%, and covers the plasticity index’s of most deepwater clays, however for clays with higher plasticity index’s, such as those reported from West Africa, further testing will be required.
7.0 MODELS FOR VERTICAL PIPE/SOIL STIFFNESS

7.1 Introduction

Pipe/soil stiffness defines the boundary condition at the touchdown point (TDP) on a steel catenary riser (SCR). If the soil stiffness chosen at the pipe/soil interface is too low the soil model will not support the SCR, however if the soil stiffness chosen is too high the soil acts as a rigid surface, which, as will be shown in Chapter 8, produces over-conservative fatigue damage at the TDP. Hence it is important to develop pipe/soil interaction models that accurately represent the pipe/soil interaction.

A realistic pipe/soil interaction model will incorporate a non-linear relationship between force and displacement that accounts for hysteresis effects and riser shape. However, most seabed models in finite element analysis programs currently use flexible or rigid surfaces to model the seabed. Therefore linear approximations of the non-linear pipe/soil interaction are required. These are developed using published literature and experimental data from the STRIDE, 2H Offshore (2002a) and CARISIMA, Marintek (2000a) Joint Industry Projects (JIPs).

7.2 Analysis and Discussion of Test Data

7.2.1 Introduction

The experimental data used to develop the pipe/soil interaction models was obtained from many sources, but primarily from the CARISIMA JIP. Details of the CARISIMA JIP pipe/soil interaction tests are given in Appendix D. The writer has used this data and conducted all of the analyses presented in this thesis.

7.2.2 Analysis Method

The test data was retrieved from the CARISIMA Matlab® files, Marintek (2001; 2002) and converted into text files containing calibrated force and displacement timetraces that can be read by Excel®. Forces and displacement timetraces were then filtered using a moving average technique to remove high frequency noise. The timetraces were then normalised using the methods presented below.
Comparisons are conducted using the normalised test data which enables the tests with different pipe diameters and/or soil properties to be assessed together and compared. The normalising method used for displacements, forces, penetration/pull-out speeds and normalised soil stiffness (in-contact cycling) are given below.

\[ \frac{z}{D} \]  
\[ N = \frac{Q}{LDS_U} \]  
\[ \frac{V}{D} \]  
\[ k_{\text{stiff}} = \frac{K}{N_cS_U} = \frac{Q_{\text{Range}}}{z_{\text{Range}}N_cS_U} \]

where

- \( z \) is the displacement or depth
- \( D \) is the external diameter of pipe
- \( N \) is the non-dimensional force
- \( Q \) is the vertical interaction force
- \( L \) is the length of the pipe
- \( S_U \) is the undrained soil shear strength
- \( V \) is the penetration or pull-out velocity
- \( k_{\text{stiff}} \) is a non-dimensional parameter for normalised soil stiffness
- \( K \) is the soil stiffness, typically taken as the secant stiffness
- \( N_c \) is the bearing capacity factor (dependent on depth)
- \( Q_{\text{Range}} \) is the vertical force range of data
- \( z_{\text{Range}} \) is the vertical displacement of data

7.2.3 Pipe/Soil Interaction Curves

An example of pipe/soil interaction curves seen in the CARISIMA test data that occur at the TDP is cycling with break-out. This is where the pipe was pushed into an existing trench after it had been pulled out of that trench and had lost contact with the
soil, as shown in Figure 7.1. This plot gives an overview of the pipe/soil interaction curves that need to be examined to define the pipe/soil interaction models. A description of the observed curves is given below.

- **Penetration** — the pipe penetrates into the soil to a depth where the soil resistance force equals the penetration force. The penetration force displacement curve follows the backbone curve. The soil deforms plastically.
- **Unloading** — the penetration force reduces to zero allowing the soil to rebound as the pipe moves upwards.
- **Pipe/soil suction** — as the pipe continues to move upwards the pipe/soil suction between the soil and the pipe causes a tensile force that resists the motion of the pipe. The adhesion force quickly increases to a maximum, then reduces to zero as the pipe moves vertically upwards and breaks out of the trench. More detail of this curve is given in the previous section.
- **Re-penetration** — the pipe re-penetrates into the existing trench that was created during the initial penetration. The re-penetration force/displacement curve has zero force when the pipe re-enters the trench, only increasing the interaction force when the pipe contacts the soil. The pipe/soil interaction force then increases until it rejoins the backbone curve at a lower depth than the previous penetration. Any further penetration follows the backbone curve.
Figure 7.1 – Pipe/soil Interaction Curves from Test Data

7.2.4 Penetration Curve

Comparisons are conducted between the normalised penetration test data and published values of bearing capacity factor, $N_c$, which have been obtained from bearing capacity theory of strip foundations. The bearing capacity factor is calculated using Skempton’s (1951) equation, viz:

$$N_c = \min \left[ 5.14 \times \left( 1 + 0.23 \sqrt{\frac{z}{B}} \right), 7.5 \right]$$

(7.5)

where

$B$ is the bearing width of the pipe (calculated as shown below).

$$B = 2\sqrt{Dz} - z^2 \text{ for depths less than } \frac{1}{2}D, \text{ else } B = D.$$  

(7.6)

A comparison between a normalised penetration force/displacement curve from a test where a 0.1016m diameter pipe was penetrating into a virgin sample of Watchet Harbour clay during the CARISIMA JIP and Skempton’s normalised soil resistance curves is given in Figure 7.2. This shows that the normalised penetration curve matches Skempton’s normalised soil resistance curve well, indicating that pipe penetration can be modelled satisfactorily using the equations for bearing capacity of
strip foundations. This demonstrates the validity of this model to SCR pipe/soil interaction applications.

Figure 7.2 – Comparison of Penetration Data with Published Values of Nc

7.2.5 Unloading Curve

It was assumed that the unloading curve and the elastic reloading curve have rotational symmetry. This allows the unloading curve to be compared to the loading curve modelled by the hyperbolic pipe/soil interaction model developed by Audibert et al (1984). This comparison was conducted to determine the validity of this model for SCR pipe/soil interaction applications. The equations of the hyperbolic pipe/soil interaction model are given below.

\[
Q = \frac{z_D}{A' + B' z_D} \\
A' = \frac{(1 - X) z_U}{Q_U} \\
B' = \frac{X}{Q_U}
\]

where

- \( Q \) is the force at displacement \( z_D \)
the ultimate force, from bearing capacity theory

a dimensionless factor that varies between 0.85 (soft clays) to 0.93 (stiff clays)

the displacement of unloading curve, sometimes called the dynamic displacement

the mobilisation distance, which is typically calculated as a function of pipe diameter as shown below.

\[ z_U = \Lambda D \]  

(7.10)

where

\[ \Lambda \]

is the normalised mobilisation distance.

The force and displacements of the hyperbolic pipe/soil interaction model were calculated using the values of \( X \) and \( \Lambda \) given by Audibert et al (1984) which are 0.85 and 0.1 respectively. The ultimate force used in the hyperbolic model was the normalised bearing capacity factor, \( N_c \), which at 0.5D penetration depth is 5.98. A comparison of the model with the unloading curve is shown in Figure 7.3. It can be seen that the hyperbolic pipe/soil interaction model matches the normalised force range, but overestimates the displacement range by approximately 400%. This indicates that the values of \( X \) and \( \Lambda \) given by Audibert et al (1984) do not model the unloading curve conservatively.

The affect of the ultimate force on the hyperbolic model was analysed. The ultimate force range was taken as the sum of the normalised penetration force and the pipe/soil suction force. The value for \( X \) was taken as 0.85. The origin of the hyperbolic pipe/soil interaction curve was then offset so that it was coincident with the origin of the unloading curve as shown in curve (2) in Figure 7.3. This shows that the hyperbolic model is a close fit to the unloading curve and the pipe/soil suction mobilisation curve.
Further analysis was conducted to compare the unloading curve with the hyperbolic model using different values of mobilisation distance. The values of $X$ and $A$ were changed to best-fit the unloading curve, as shown in Figure 7.4. By observation the best fit was observed with values of $X$ and $A$ of 0.85 and 0.04 respectively. However, this solution of the model overshoots the displacement where the pipe/soil suction curve crosses the zero force line by approximately $0.015D$ and is thought to be an unconservative representation of the unloading curve. A second comparison was conducted where the displacement of the pipe/soil interaction model matched the pipe/soil suction zero crossing point. By observation this value of $A$ was found to be $0.025$ and gave a more conservative hyperbolic pipe/soil interaction model.

Analysis was conducted comparing the hyperbolic model with different values of normalised mobilisation distance with many unloading curves. The results show that for all penetration depths ($0.25D$, $0.5D$, $1.0D$ and $1.5D$) the typical value of $A$, normalised mobilisation distance, was around 0.025, the maximum being 0.037, the minimum 0.018 and the median 0.27. This is shown in a close up of an unloading curve in Figure 7.5. A similar investigation was conducted into the approximate distance between the end of the penetration curve and the start of the maximum
pipe/soil suction curve. From inspection of the test data $\Lambda$ was determined to be 0.1. This is summarised in Figure 7.5.

Figure 7.4 – Close Up of Unloading Curves with Hyperbolic Model

Figure 7.5 – Close up of Unloading Curve
7.2.6 In-Contact Cycling

In-contact cycling test data from the CARISIMA JIPs was examined to determine values of small-displacement dynamic soil stiffness. The tests, which are described in Appendix D, used two pipes of different diameters that were penetrated up to one and a half diameters into either the Onsoy or Watchet Harbour clays and oscillated for up to 100 cycles by force control.

The force and displacement timetraces were filtered using a moving average filter to remove high frequency noise within the data as shown in Figure 7.6. The timetraces were then truncated so that only whole numbers of peak-to-peak force cycles were used in subsequent data analysis as shown in Figure 7.7. These were then plotted using vertical force and horizontal displacement axis as a typical force/displacement plot as shown in Figure 7.8.

The next stage of the analysis was to determine a value of linear soil stiffness that represented the cyclic data and could be used to calculate a value of $k_{stiff}$. Since the tests were force controlled the linear stiffness was calculated using the least-squares method assuming that any error occurred in the displacement values. The cyclic data is plotted with displacement in the vertical axis and the force in the horizontal axis, Figure 7.9. The inverse of the gradient of the least-squares linear fit line divided by the pipe length is the linear soil stiffness per metre length. The linear soil stiffness is then divided by $N_c$ and $S_u$ to calculate $k_{stiff}$ as described previously. An example of these calculations for CARISIMA II, test 1-2 are shown in Table 7.1. All of the in-contact cyclic test data is processed using this method.

<table>
<thead>
<tr>
<th>Data Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gradient of least squares fit from displacement/force curve</td>
<td>1.158x10^-6 m/N</td>
</tr>
<tr>
<td>Linear soil stiffness from inverse of gradient</td>
<td>864 kN/m</td>
</tr>
<tr>
<td>Linear soil stiffness per metre</td>
<td>2125 kN/m/m</td>
</tr>
</tbody>
</table>
A summary of the soil stiffness factors for each in-contact cyclic test examined is given in Table 7.2 and Table 7.3. This shows that within the tests examined the maximum soil stiffness factor was 183 while the lowest was 55.

**Figure 7.6** – Filtering Force and Displacement Timetraces

**Figure 7.7** – Filtering Force and Displacement Timetraces
Figure 7.8 – Cyclic Soil Stiffness Force Displacement Curve

Figure 7.9 – Cyclic Soil Stiffness Force Displacement Curve
<table>
<thead>
<tr>
<th>Test Number</th>
<th>Pipe Diameter (m)</th>
<th>Number of Cycles</th>
<th>Trench Depth, z/D</th>
<th>Undrained Shear Strength (kPa)</th>
<th>Gradient, D/F</th>
<th>Soil Stiffness (kN/m/m)</th>
<th>kstiff (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CI - 5</td>
<td>0.1016</td>
<td>1</td>
<td>0.5</td>
<td>2.48</td>
<td>1.02x10^-6</td>
<td>2412.4</td>
<td>162.8</td>
</tr>
<tr>
<td></td>
<td>0.1016</td>
<td>10</td>
<td>0.5</td>
<td>2.48</td>
<td>1.41x10^-6</td>
<td>1745.1</td>
<td>117.8</td>
</tr>
<tr>
<td></td>
<td>0.1016</td>
<td>100</td>
<td>0.5</td>
<td>2.48</td>
<td>1.60x10^-6</td>
<td>1537.9</td>
<td>103.8</td>
</tr>
<tr>
<td>CI - 7a</td>
<td>0.1016</td>
<td>1</td>
<td>0.5</td>
<td>2.22</td>
<td>1.25x10^-6</td>
<td>1968.5</td>
<td>148.4</td>
</tr>
<tr>
<td></td>
<td>0.1016</td>
<td>10</td>
<td>0.5</td>
<td>2.22</td>
<td>1.15x10^-6</td>
<td>2139.7</td>
<td>161.3</td>
</tr>
<tr>
<td></td>
<td>0.1016</td>
<td>100</td>
<td>0.5</td>
<td>2.22</td>
<td>1.43x10^-6</td>
<td>1720.7</td>
<td>129.7</td>
</tr>
<tr>
<td>CI - 7b</td>
<td>0.1016</td>
<td>1</td>
<td>0.5</td>
<td>2.65</td>
<td>3.52x10^-6</td>
<td>699.0</td>
<td>180.0</td>
</tr>
<tr>
<td></td>
<td>0.1016</td>
<td>10</td>
<td>0.5</td>
<td>2.65</td>
<td>4.26x10^-6</td>
<td>577.6</td>
<td>148.7</td>
</tr>
<tr>
<td></td>
<td>0.1016</td>
<td>100</td>
<td>0.5</td>
<td>2.65</td>
<td>5.43x10^-6</td>
<td>453.2</td>
<td>116.7</td>
</tr>
<tr>
<td>CI - 8</td>
<td>0.1016</td>
<td>1</td>
<td>0.67</td>
<td>2.40</td>
<td>4.36x10^-7</td>
<td>2617.0</td>
<td>182.5</td>
</tr>
<tr>
<td></td>
<td>0.1016</td>
<td>10</td>
<td>0.67</td>
<td>2.40</td>
<td>4.66x10^-7</td>
<td>2448.6</td>
<td>170.7</td>
</tr>
<tr>
<td></td>
<td>0.1016</td>
<td>100</td>
<td>0.67</td>
<td>2.40</td>
<td>4.97x10^-7</td>
<td>2295.8</td>
<td>160.1</td>
</tr>
<tr>
<td>CI - 9</td>
<td>0.2191</td>
<td>1</td>
<td>0.5</td>
<td>2.40</td>
<td>4.36x10^-7</td>
<td>2617.0</td>
<td>182.5</td>
</tr>
<tr>
<td></td>
<td>0.2191</td>
<td>10</td>
<td>0.5</td>
<td>2.40</td>
<td>4.66x10^-7</td>
<td>2448.6</td>
<td>170.7</td>
</tr>
<tr>
<td></td>
<td>0.2191</td>
<td>100</td>
<td>0.5</td>
<td>2.40</td>
<td>4.97x10^-7</td>
<td>2295.8</td>
<td>160.1</td>
</tr>
<tr>
<td>CI - 11</td>
<td>0.1016</td>
<td>1</td>
<td>0.5</td>
<td>2.06</td>
<td>1.61x10^-6</td>
<td>1528.3</td>
<td>124.2</td>
</tr>
<tr>
<td></td>
<td>0.1016</td>
<td>10</td>
<td>0.5</td>
<td>2.06</td>
<td>1.91x10^-6</td>
<td>1288.3</td>
<td>104.7</td>
</tr>
<tr>
<td></td>
<td>0.1016</td>
<td>100</td>
<td>0.5</td>
<td>2.06</td>
<td>2.63x10^-6</td>
<td>935.6</td>
<td>76.0</td>
</tr>
<tr>
<td>CI - 12</td>
<td>0.2191</td>
<td>1</td>
<td>0.5</td>
<td>2.55</td>
<td>4.59x10^-7</td>
<td>2485.9</td>
<td>163.1</td>
</tr>
<tr>
<td></td>
<td>0.2191</td>
<td>10</td>
<td>0.5</td>
<td>2.55</td>
<td>4.69x10^-7</td>
<td>2432.9</td>
<td>159.7</td>
</tr>
<tr>
<td></td>
<td>0.2191</td>
<td>100</td>
<td>0.5</td>
<td>2.55</td>
<td>4.69x10^-7</td>
<td>2432.9</td>
<td>159.7</td>
</tr>
<tr>
<td>CI - 13</td>
<td>0.1016</td>
<td>1</td>
<td>0.5</td>
<td>2.02</td>
<td>2.39x10^-6</td>
<td>1029.6</td>
<td>85.3</td>
</tr>
<tr>
<td></td>
<td>0.1016</td>
<td>10</td>
<td>0.5</td>
<td>2.02</td>
<td>3.12x10^-6</td>
<td>788.7</td>
<td>65.3</td>
</tr>
<tr>
<td></td>
<td>0.1016</td>
<td>100</td>
<td>0.5</td>
<td>2.02</td>
<td>3.81x10^-6</td>
<td>645.8</td>
<td>53.5</td>
</tr>
<tr>
<td>CI - 14</td>
<td>0.1016</td>
<td>1</td>
<td>0.5</td>
<td>0.81</td>
<td>3.07x10^-6</td>
<td>801.5</td>
<td>165.6</td>
</tr>
<tr>
<td></td>
<td>0.1016</td>
<td>10</td>
<td>0.5</td>
<td>0.81</td>
<td>4.02x10^-6</td>
<td>612.1</td>
<td>126.5</td>
</tr>
<tr>
<td>CI - 5</td>
<td>0.1016</td>
<td>100</td>
<td>0.5</td>
<td>0.81</td>
<td>5.54x10^-6</td>
<td>444.2</td>
<td>91.8</td>
</tr>
</tbody>
</table>

Table 7.2 – Cyclic Tests Matrix, CARISIMA I Tests
The effect of the cycling rate, or vertical riser velocity, on the soil stiffness factor is shown in Figure 7.10. This data shows that as the cyclic rate doubles the soil stiffness factor, hence soil stiffness, increases by about 9%.

The effect of repeated cycling of the pipe in the soil on the stiffness factor, \( k_{\text{stiff}} \), is shown in Figure 7.11. This data shows that as the number of cycles increases, the soil stiffness factor (and hence the soil stiffness) reduces. Typically, after ten cycles the soil stiffness factor is approximately 80% of the first cycle while after 100 cycles the soil stiffness factor is approximately 63% of the first cycle. This is summarised in
Table 7.4. The exception to this trend was observed in tests 7a, 9 and 12. Closer examination of these tests showed that the displacement timetraces contain noise that was not removed by the moving average filtering technique. The noise alters the results of the least squares fit that influence the calculation of the soil stiffness factor.

![Soil Stiffness with Cyclic Load](image)

**Figure 7.10 – Stiffness Factor with Cycle Rate**

<table>
<thead>
<tr>
<th>Cycle</th>
<th>Average Reduction in the Soil Stiffness Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0%</td>
</tr>
<tr>
<td>10</td>
<td>20%</td>
</tr>
<tr>
<td>100</td>
<td>37%</td>
</tr>
</tbody>
</table>
7.2.7 Cycling with Breakout

The method for calculating the normalised soil stiffness was also used to assess the pipe/soil stiffness in large displacement cycling, where the riser was pulled out of, and then re-penetrated into, the soil. Two normalised soil stiffnesses were calculated, the first from the unloading curve and the second using the combined unloading and pipe/soil suction mobilisation curves. The normalised soil stiffnesses from this assessment of the test data is summarised in Table 7.5 and shows that the values of $k_{stiff}$ are between 1.5 and 119 with a mean of 20.2 for the unloading curve and between 0.1 to 11.2 with a mean of 5.5 for the combined unloading and pipe/soil suction curve.
Table 7.5 – Summary of $k_{\text{s}t\text{iff}}$ from Cycling with Break Out

<table>
<thead>
<tr>
<th>Test</th>
<th>Group</th>
<th>Cycle</th>
<th>Velocity (mm/s)</th>
<th>Period (s)</th>
<th>Norm Force (-)</th>
<th>$S_u$ (kPa)</th>
<th>Depth z/D</th>
<th>Unloading $k_{\text{s}t\text{iff}}$ (-)</th>
<th>Unloading + Suction $k_{\text{s}t\text{iff}}$ (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-I 8-1</td>
<td>a</td>
<td>1</td>
<td>10</td>
<td>300</td>
<td>0.83</td>
<td>3.17</td>
<td>0.833</td>
<td>41.8</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>1</td>
<td>80</td>
<td>300</td>
<td>0.83</td>
<td>3.17</td>
<td>0.833</td>
<td>53.3</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td>c</td>
<td>1</td>
<td>5</td>
<td>300</td>
<td>0.83</td>
<td>3.17</td>
<td>0.833</td>
<td>11.6</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>d</td>
<td>1</td>
<td>1</td>
<td>300</td>
<td>0.83</td>
<td>3.17</td>
<td>0.833</td>
<td>7.4</td>
<td>3.6</td>
</tr>
<tr>
<td></td>
<td>e</td>
<td>1</td>
<td>10</td>
<td>300</td>
<td>0.83</td>
<td>3.17</td>
<td>0.833</td>
<td>15.3</td>
<td>2.3</td>
</tr>
<tr>
<td></td>
<td>e</td>
<td>2</td>
<td>10</td>
<td>300</td>
<td>0.83</td>
<td>3.17</td>
<td>0.833</td>
<td>8.1</td>
<td>3.4</td>
</tr>
<tr>
<td>C-I 8-2</td>
<td>a</td>
<td>1</td>
<td>10</td>
<td>300</td>
<td>0.74</td>
<td>3.55</td>
<td>0.737</td>
<td>5.5</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>1</td>
<td>80</td>
<td>300</td>
<td>0.74</td>
<td>3.55</td>
<td>0.737</td>
<td>13.0</td>
<td>0.4</td>
</tr>
<tr>
<td></td>
<td>c</td>
<td>1</td>
<td>5</td>
<td>300</td>
<td>0.74</td>
<td>3.55</td>
<td>0.737</td>
<td>2.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>d</td>
<td>1</td>
<td>1</td>
<td>300</td>
<td>0.74</td>
<td>3.55</td>
<td>0.737</td>
<td>1.7</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>e</td>
<td>1</td>
<td>10</td>
<td>300</td>
<td>0.74</td>
<td>3.55</td>
<td>0.737</td>
<td>2.2</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>e</td>
<td>2</td>
<td>10</td>
<td>300</td>
<td>0.74</td>
<td>3.55</td>
<td>0.737</td>
<td>1.5</td>
<td>0.5</td>
</tr>
<tr>
<td>C-I 9-1</td>
<td>a</td>
<td>1</td>
<td>10</td>
<td>300</td>
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</table>

7.3 **Overview of Vertical Pipe/Soil Interaction**

Using the observations of the test data an example of the development of an in-contact cycling pipe/soil interaction curve is presented in Figure 7.12. The right column shows the relationship between the backbone curve, the maximum soil resistance force to pipe penetration at a given depth in a virgin soil, and the pipe/soil interaction curve (the force/displacement relationship) of a pipe moving through the soil. The left column shows the vertical motion of the pipe associated with the pipe/soil interaction curve in the right column, as described below:

1. The pipe is suspended above a virgin soil.
2. The pipe penetrates into the soil, plastically deforming it. The pipe/soil interaction curve follows the backbone curve.
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<th>Soil Resistance Force per unit Length, Q</th>
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<td>Penetrate Pipe using Self Weight</td>
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<td><strong>3</strong></td>
<td>Pipe Moves Upwards</td>
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<td><strong>4</strong></td>
<td>Pipe Moves into Contact with Soil</td>
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<td><strong>5</strong></td>
<td>Pipe is Pushed Further into Soil</td>
<td><img src="image6" alt="Soil Resistance Force per unit Length, Q" /></td>
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</tbody>
</table>

Figure 7.12 – Illustration of Pipe/soil Interaction
3. The pipe moves upwards and the soil responds elastically. The pipe/soil interaction curve breaks away from the backbone curve, and the force reduces over a small displacement.

4. The pipe again penetrates the soil, deforming it elastically. The pipe/soil interaction curve follows an elastic loading curve similar to the previous elastic unloading curve (step 3).

5. The pipe again penetrates into the soil, plastically deforming it. The pipe/soil interaction curve rejoins and follows the backbone curve.

The other type of pipe/soil interaction that can occur at the TDP is cycling with break-out, where the pipe is pushed into an existing trench after it has been pulled out of the trench and lost contact with the soil. Typical force/displacement curves during cyclic loading are presented in Figure 7.13.

![Figure 7.13 – Summary of Force/displacement Curves](image)

From observations of the test data and using the pipe/soil interaction examples above, the backbone curve and three types of pipe/soil stiffnesses are identified that are important in SCR TDP interaction. These are:
• Backbone curve, which is used to model the initial penetration of the pipe into a virgin soil and the maximum force/displacement curve for the pipe/soil interaction models.

• Static pipe/soil interaction, which is used to model the initial penetration of the riser into the soil

• Large displacement pipe/soil interaction, which is used to model quasi-static loading where the riser is subject to large and/or long period displacements.

• Cyclic pipe/soil interaction, which is used for dynamic TDP motions.

The pipe/soil interaction models developed by the writer to represent these stiffnesses are described in the following section.

7.4 Development of Pipe/Soil Interaction Models

7.4.1 Backbone Curve

The backbone curve is used to model the penetration of the riser into a virgin soil. It is also a measure of how the maximum compressive soil resistance force per unit length varies with depth below the mud line. The backbone curve is derived from the bearing capacity theory for strip foundations and has been investigated and discussed in many papers, including Terzaghi (1943), Skempton (1951), Meyerhof (1963) and Bostrom et al (1998). The equation for the backbone curve is given below.

\[ Q_u = q_u B \]  \hspace{1cm} (7.11)

where

\( q_u \) the ultimate bearing pressure, and is calculated using the equation below.

\[ q_u = N_c S_u + \gamma z \]  \hspace{1cm} (7.12)

where

\( S_u \) the undrained shear strength of soil, and a function of the penetration depth, see equation (7.13) below.

\( \gamma \) the unit weight of soil

\[ S_u = S_{u0} + S_{ug} z \]  \hspace{1cm} (7.13)
where

\[ S_{U0} \quad \text{undrained shear strength at the mudline} \]

\[ S_{UG} \quad \text{gradient of the undrained shear strength with depth} \]

For most practical purposes the undrained shear strength is calculated at the invert of the pipe. The \( \gamma z \) term, which represents the weight of the backfill material, in the bearing capacity equation is applicable to pipelines that are not backfilled or buried naturally.

### 7.4.2 Static Pipe/Soil Interaction Model

For static FE analysis the pipe/soil interaction curve is required to model the riser shape. An accurate solution is to model the backbone curve using non-linear springs. This approach is suited to static analysis where the riser does not move, however in quasi-static analysis, where large riser movements are modelled, the axial and lateral pipe/soil forces are also required. A better solution is to use a flexible surface that accounts for lateral and axial friction and changes in the riser/soil contact length due to TDP zipping. Unfortunately most state-of-the-art riser packages use linear flexible surfaces that have a single value for soil stiffness, indicating that for FE analysis the non-linear backbone curve needs to be linearised. The method used is the secant stiffness as this accurately models the penetration at the TDP, where the stresses are greatest. A tangent model could be too soft at larger depths and too stiff at shallow depth. An example of the secant stiffness model is given in Figure 7.14.
Figure 7.14 – Example of Dynamic Soil Stiffness

The static pipe/soil stiffness is calculated using the secant stiffness formula that is the force at depth divided by that depth, as shown below.

\[
K_s = \frac{Q_u}{z_p} \tag{7.14}
\]

where

- \( K_s \) static pipe/soil stiffness
- \( z_p \) penetration depth

The force used is the TDP reaction force which can be conservatively calculated using the equation given by Pesce et al (1998) for a rigid seabed. The penetration depth is then calculated from the backbone curve, the equations for which are given previously. Combining the two equations results in a relationship for the ultimate vertical force as shown below.

\[
Q_u = (N_c S_u + g z_p) B = m_s g \sqrt{\frac{EI}{H}} \tag{7.15}
\]

where

- \( m_s \) the mass per unit length of pipe
- \( g \) the acceleration due to gravity
- \( E \) Young’s modulus of steel
- \( I \) the second moment of inertia of the riser
- \( H \) the tension in the riser at the TDP
The static pipe/soil stiffness at the TDP is calculated by division of the penetration depth as shown below.

\[ K_S = \frac{Q_U}{z_p} = \left( \frac{N_c S_u + \gamma}{z_p} \right) B = \frac{mg}{z_p} \sqrt{\frac{EI}{H}} \]  

(7.16)

Due to the non-linear relationship between \( z_p \) and \( Q_U \) because \( N_c, S_u \) and \( B \) are all functions of penetration depth, the exact solution to the above equation to determine the static pipe/soil stiffness is complex. A more realistic method to determine the static pipe/soil stiffness is to calculate the TDP reaction force then iterate to determine the penetration depth, and hence pipe/soil stiffness. However, an estimation of the static penetration depth and consequently pipe/soil stiffness can be made using the following assumptions:

- The bearing width is equal to the riser diameter, \( B = D \)
- The penetration is shallow, less than 5D, so that the \( \gamma z \) term in the bearing capacity equation is assumed small compared to \( N_c S_u \) and ignored.
- The undrained shear strength of the soil at the surface, \( S_{u0} \), is virtually zero so only the undrained shear strength gradient, \( S_{ug} \), is significant.
- The bearing capacity factor, \( N_c \), is assumed to be constant with depth and taken as 6.

Rearranging equation (7.15) and using the assumptions above produces the following equation for an estimate of the penetration depth.

\[ z_p = \frac{mg}{6S_{ug}D} \sqrt{\frac{EI}{H}} = \frac{Q_U}{6S_{ug}D} \]  

(7.17)

Which when substituted into equation (7.16) for static pipe/soil stiffness simplifies to

\[ K_S = 6S_{ug}D \]  

(7.18)

This shows that the static pipe/soil interaction stiffness is proportional to the riser diameter and the undrained soil shear strength. A more conservative estimate for static pipe/soil stiffness is to add the undrained shear strength at the surface back into the equation giving:
The reaction force used in equation (7.15) is observed to cancel out of equation (7.18) and consequently equation (7.18) is thought to calculate the static seabed stiffness along the riser length.

7.4.3 Large Displacement Pipe/soil Interaction Model

Large displacement pipe/soil interaction is used to model the soil stiffness during large riser motions where the pipe breaks away from the soil. These motions occur when the riser is pulled from the seabed trench or during long periods of second order motions due to vessel drift. The pipe/soil interaction curves that represent the large displacement pipe/soil interaction model are the unloading curve and the pipe/soil suction mobilisation curve (the part of the pipe/soil suction curve from the unloading curve to the maximum pipe/soil suction force).

As in the static pipe/soil interaction model the best linear representation of the non-linear unloading curve is the secant stiffness. However, unlike the static model the secant stiffness is between the force at penetration depth and the displacement of the soil swelling caused by unloading. This is shown in Figure 7.15. Using this an equation for the large displacement soil stiffness can be written as follows.

\[ K_L = \frac{Q_u}{\Lambda D} = \frac{q_u B}{\Lambda D} \]  \hspace{1cm} (7.20)

where

- \( K_L \) is the large displacement pipe/soil interaction stiffness

Observations of SCR trenches during the STRIDE JIP concluded that trench depths (and hence penetration depths) are greater than \( \frac{1}{2}D \). This indicates that for soil stiffness calculations, other than the static stiffness, the bearing width can be assumed to be equal to the pipe diameter. This simplifies equation (7.20) to the following.

\[ K_L = \frac{q_u D}{\Lambda D} = \frac{1}{\Lambda} q_u \]  \hspace{1cm} (7.21)
This shows that the large displacement pipe/soil interaction stiffness is not dependant on the riser diameter and is inversely proportional to the normalised mobilisation distance. Therefore, at a given penetration depth, all risers will have the same value of pipe/soil stiffness.

Soil Resistance, Q

\[ K_L = \frac{1}{\Lambda} \left( N_c S_U + \gamma z \right) \]  

(7.22)

If the trench depth is assumed to be shallow then the \( \gamma z \) term is small and equation (7.22) simplifies to

\[ K_L = \frac{1}{\Lambda} N_c S_U \]  

(7.23)

This equation is similar to published equations for the Young’s modulus of a clay soil that has been calculated using plate bearing tests by D’Appolonia et al (1971). This equation has the form.

\[ E = \beta S_U \]  

(7.24)

where

\( \beta \) a dimensionless parameter determined from experiments, and is 400 for cycling (Smits, 1980) and between 800 and 2500 for static loading on clay soils (D’Appolonia et al, 1971).
Observations of test data show that the normalised mobilisation distance is approximately equal to 0.025. Using this data equation (7.23) simplifies to the following.

\[ K_L = 40N_C S_U \]  \hspace{1cm} (7.25)

It is possible that the above equation is conservative and a more realistic value for large displacement dynamic soil stiffness should use the displacement from the backbone curve to the pipe/soil suction peak. Using observations of the STRIDE and CARISIMA test data this normalised mobilisation distance is found to be 0.1. Conservatively, for short consolidation times, the pipe/soil suction force can be assumed to equal the penetration force from the backbone curve. This modifies equation (7.23) and simplifies to the following.

\[ K_L = 2\frac{1}{\Lambda} N_C S_U = 20N_C S_U \]  \hspace{1cm} (7.26)

### 7.4.4 Cyclic Pipe/Soil Interaction Model

Cyclic pipe/soil interaction is used to model the soil stiffness when the TDP is cycled in the soil. Both day-to-day and storm vessel motions can cause this type of interaction. For large cyclic displacements, where the riser brakes away from the soil, the pipe/soil interaction stiffness is based on the large displacement model derived previously. For small cyclic displacements, where the riser is continually in-contact with the soil the large displacement pipe/soil interaction model needs to be modified to incorporate the hyperbolic pipe/soil interaction model.

The first step is to generalise the large displacement pipe/soil interaction model so that the normalised mobilisation distance is replaced by a pipe/soil stiffness factor, \( k_{\text{stiff}} \). The pipe/soil interaction model now takes the following form.

\[ K_D = k_{\text{stiff}} q_U = k_{\text{stiff}} N_C S_U \]  \hspace{1cm} (7.27)

where

\[ K_D \] is dynamic pipe/soil interaction stiffness
The hyperbolic model scales the cyclic loading force/displacement curve using the ultimate penetration force at a given depth from the backbone curve and an assumed unloading/reloading displacement. The hyperbolic model is generally given in the following shown previously in equations (7.7), (7.8) and (7.9). However, the hyperbolic pipe/soil interaction model can be expanded and rearranged into a more useful form as shown below.

\[ Q = \frac{z_D}{(1 - X)\Lambda D + Xz_D}Q_U \]  \hspace{1cm} (7.28)

Using this form of the hyperbolic pipe/soil interaction equation, an expression for \( k_{\text{stiff}} \) can be derived, namely.

\[ k_{\text{stiff}} = \frac{1}{\Lambda(1 - X) + \frac{Xz_D}{D}} \]  \hspace{1cm} (7.29)

The maximum value of \( k_{\text{stiff}} \), which results in the most conservative value for soil stiffness is at the origin of the hyperbolic pipe/soil interaction model where \( z_D = 0.0 \). This results in the following equation for \( k_{\text{stiff,MAX}} \).

\[ k_{\text{stiff,MAX}} = \frac{1}{\Lambda(1 - X)} \]  \hspace{1cm} (7.30)

Assuming that the seabed is a soft clay soil, \( X = 0.85 \), and that \( \Lambda = 0.025 \) then the maximum value of \( k_{\text{stiff}} \) is 267. The minimum value of \( k_{\text{stiff}} \) occurs when the dynamic displacement is equal to the mobilisation distance, \( z_D = \Lambda D \). This results in the following formula for \( k_{\text{stiff,MIN}} \).

\[ k_{\text{stiff,MIN}} = \frac{1}{\Lambda} \]  \hspace{1cm} (7.31)

Assuming that the normalised mobilisation distance is 0.025 then the minimum value for \( k_{\text{stiff}} \) is 40, which is the same as calculated by the conservative large displacement pipe/soil interaction model.

The other important factors relating to pipe/soil interaction stiffness are hysteresis and penetration speed. When these factors are included, the dynamic pipe/soil interaction equation becomes;

\[ K_D = k_H k_F k_{\text{stiff}} N_C S_U \]  \hspace{1cm} (7.32)
where

\[ k_H \quad \text{a factor accounting for hysteresis effects} \]
\[ k_F \quad \text{a factor accounting for the frequency of the oscillations} \]

Equations for the factors can be constructed using the test data examined previously. The hysteresis factor, \( k_H \), can be modelled using a logarithmic function that is based on the data given in Table 7.4, the displacement controlled curves from Dunlap et al (1990) and cyclic load data from Andersen (1991). These data sets are given in Figure 7.16 and show that over the first 100 cycles the cyclic reduction factor reduces to approximately 2/3. After this the Dunlap et al (1990) data increase to 0.8 at 3000 cycles while Andersen (1991) continues to reduce to 0.2 at 1000 cycles. This indicates that there is conflicting evidence and more investigation is required to determine the cyclic reduction factors. However, from the data examined, a cyclic reduction factor model is proposed that is based on a power law relationship between the number of cycles and the cyclic reduction factor as shown below, which is the same as the cyclic reduction factor proposed by Idriss et al (1978). It is also recognised that, with the current data available, a minimum cyclic reduction factor of 0.6 is proposed as a conservative lower bound:

\[ k_H = \text{Max}(C^{-0.1}, 0.6) \]  (7.33)

where

\( C \quad \text{is the number of cycles that have recently occurred.} \)
An equation for the frequency factor is developed based on the tests data that showed a 9% increase in soil stiffness as the cyclic frequency doubles. This is converted into the following equation:

$$k_F = 0.8f + 0.92$$

(7.34)

where

$$f$$ is the frequency of oscillation

However it is important to note that this equation may be subject to diameter scaling which will alter the values of $k_F$ for the given frequencies. It is expected that the derived relationship will be similar to equation (6.2) where the velocity factor is proportional to a power of the pull-out velocity divided by pipe diameter. However, the derived empirical parameters are smaller. Further work is required to establish this trend for different pipe diameters and different soils.

7.5 Comparison of Dynamic Pipe/soil Interaction Model with Test Data

The normalised pipe/soil stiffness model was compared to measured normalised pipe/soil stiffnesses to show that the dynamic pipe/soil interaction model was conservative. A series of hyperbolic pipe/soil interaction model trend lines were
created where $X$ is 0.85 and $\Lambda$ is either 0.1, 0.04 and 0.025 as shown in Figure 7.17. The measurements of pipe/soil interaction stiffness are plotted on top of these trend lines. The comparison shows that the model with a $\Lambda$ of 0.025 is conservative, giving values of normalised stiffness that are greater than 90% of the measured values. A value of $\Lambda$ of 0.06 gives a lower bound normalised stiffness to 90% of the in-contact cycling data.

Comparing the normalised soil stiffness data (calculated using the breakout data) with the hyperbolic pipe/soil interaction models, the following conclusions are made. For all dynamic displacements above $0.001(z_d/D)$ the hyperbolic model where $\Lambda$ is 0.1 is conservative for all tests. For dynamic displacements below $0.001(z_d/D)$ the hyperbolic model where $\Lambda$ is 0.025 is conservative. For the majority of tests where large dynamic displacements occurred (greater than $0.02(z_d/D)$) the values of $k_{stiff}$ are less than 25. If the normalised soil stiffness is calculated using the combined unloading and the pipe/soil suction curves the value of $k_{stiff}$ is below 12. This shows that, for this data, the hyperbolic model where $X$ is 0.85 and $\Lambda$ is 0.1 is conservative.

![Figure 7.17 – Stiffness Factor with Dynamic Displacement](image-url)
7.6 Summary and Conclusions

Pipe/soil stiffness models the boundary conditions of the SCR with the seabed. Pipe/soil interaction data from the STRIDE and CARISIMA JIPs has been examined and a series of pipe/soil interaction curves identified, which are penetration, unloading, pipe/soil suction, re-penetration and cycling loading. Comparisons between tests have been conducted using normalisation parameters that account for pipe diameter, pipe length, pipe velocity, undrained shear strength of the soil and trench depth. Additionally $k_{stiff}$, a non-dimensional parameter developed by the writer, is used for normalising cyclic pipe/soil interaction data.

The applicability of existing pipe/soil interaction models is investigated by comparing them with the pipe/soil interaction data. The conclusions from these comparisons are summarised below.

- Comparisons between the penetration data and Skemtions (1951) bearing capacity equations show good correlation.
- Examination of the unloading curves with the hyperbolic model shows that the model compares well with a normalised mobilisation distance, $\Lambda$, of 2.5%.

A method for processing the force controlled in-contact cyclic test data was developed and presented. The test data was filtered to remove noise and then analysed using a least squares method. The linear soil stiffness was then converted to a value for $k_{stiff}$, which was found to vary between 100 and 180 for small ($<0.005 \frac{z_D}{D}$) dynamic displacements, depending on the cyclic rate and the number of previous cycles. The same method was used to process the cycling data with break out, and $k_{stiff}$ was found to be below 25 for dynamic displacements above $0.005 \frac{z_D}{D}$ and decrease with increasing dynamic displacement.

The observations from the penetration, unloading and cyclic tests were then used to create a series of state-of-the-art vertical pipe/soil interaction models for use in SCR analysis. These models include a backbone curve model, static penetration, large displacement pipe/soil interaction and cyclic pipe/soil interaction. The models have
been constructed for use in finite element analysis software, especially those codes developed for marine riser applications.

The static pipe/soil interaction model used the rigid seabed reaction force and the backbone curve to estimate the minimum soil stiffness for static analysis.

The large displacement model was used to model the pipe/soil stiffness where the riser breaks away from the soil. The model developed was similar to plate bearing test models by D’Appolonia et al (1971), but include the depth factor, Nc, to account for trenches.

The cyclic pipe/soil interaction models are similar to the large displacement models but include the k_{stiff}, k_H and k_F parameters to account for dynamic displacement distance, hysteresis and the frequency of the cycles respectively. Using published literature and the results from the CARISIMA tests equations have been developed to calculate appropriate values of k_{stiff}, k_H and k_F. The cyclic model is also calibrated using the available test data and is shown to be conservative.
8.0 CLOSED FORM SEABED INTERACTION MODELS

8.1 Introduction

Two of the reasons for conducting analysis on steel catenary risers (SCR) are to check that the maximum stress in the riser is below the yield stress of the material, and to calculate the fatigue life. Generally, finite element analysis (FEA) programs, such as FLEXCOM-3D (MCS, 2004) and ANSYS (ANSYS Inc, 2000) are used. These show how a model of the riser responds to a series of static and dynamic loads and boundary conditions, including pipe/soil interaction. However, an understanding of the system mechanics and relationships can be obscured, as FEA produces answers and not the underlying equations. Consequently another method of SCR analysis is required that helps to develop an understanding of the mechanics of the riser/soil interaction problem. This may be achieved through the use of closed form solutions.

8.2 SCR Analysis

8.2.1 Catenary Equations

The forces in the catenary zone of a SCR can be modelled using the catenary equations. These were first proposed by Leibniz (1691a, 1691b) and furthered developed by Timoshenko (1965), details of which are given in Appendix A. This Appendix also shows that the maximum von Mises stress occurs at either the vessel/riser connection or the touchdown point (TDP). The stress in the riser/vessel connection is dominated by tension, as the curvature in the top section of the riser is small. Near the TDP the tension is low (compared to the riser/vessel connection) while the curvature is high, indicating that the von Mises stress at the TDP is dominated by the bending moment. From the solutions of the catenary equations derived in Appendix A the static bending moment at the TDP can be calculated using the equation below.

\[ M_{\text{TDP}} = \frac{m_s g}{H} EI \]  \hspace{1cm} (8.1)

where

- \( M_{\text{TDP}} \) is the bending moment at the TDP,
- \( m_s \) is the submerged mass per unit length of the riser,
g is the acceleration due to gravity,

H is the tension in the riser at the TDP,

E is Young’s modulus,

I is the second moment of inertia.

The other force calculated using the catenary equations (providing that the top angle, \( \alpha_{\text{TOP}} \), height of attachment point, \( z_A \), and submerged mass per unit length, \( m_s \), of the riser are known) is the horizontal (or TDP) tension, H. However the catenary equations incorrectly calculate the shear force at the TDP to be zero. For a rigid seabed the maximum shear force can be calculated by the equation given by Pesce et al (1998) and Palmer (2000) as shown below.

\[
R_c = m_s g \sqrt{\frac{EI}{H}}
\]  

where

Rc is the rigid seabed reaction force at the TDP

This can be written as a function of the TDP bending moment as shown below.

\[
R_c = \sqrt{\frac{H}{EI}} \frac{M_{\text{TPD}}}{M_{\text{TD}}}, \quad \frac{1}{\lambda_L} = \frac{M_{\text{TD}}}{M_{\text{TPD}}}
\]  

where

\( \lambda_L \) is the flexural length parameter, as defined by Pesce et al (1998).

### 8.2.2 Stress Analysis

Ultimately one of the most important parameters in riser design is the overall stress in the riser. This is usually given as the von Mises stress, which combines the axial, hoop and radial stresses as shown below:

\[
\sigma_{vM} = \frac{1}{\sqrt{2}} \sqrt{(\sigma_R - \sigma_\theta)^2 + (\sigma_\theta - \sigma_A)^2 + (\sigma_A - \sigma_R)^2}
\]

where

\( \sigma_{vM} \) is the von Mises stress

\( \sigma_R \) is the radial stress

\( \sigma_\theta \) is the hoop stress
\( \sigma_A \) is the axial stress

As stated previously the stress at the TDP is dominated by the bending moment, and hence the axial stress calculated from:

\[
\sigma_d = \frac{T}{A_s} \pm \frac{My}{I} \tag{8.5}
\]

where

- \( T \) is the tension at the location being considered
- \( A_s \) is the cross-sectional area of the riser pipe
- \( M \) is the bending moment at the location being considered
- \( y \) is the distance from the neutral axis to the extreme fibre

The maximum axial stress at the TDP can then be calculated using the following formula.

\[
\sigma_{A,TDP} = \frac{H}{A_s} + \frac{m_y g E D_o}{2H} \tag{8.6}
\]

where

- \( D_o \) is the outer diameter of the riser

8.2.3 Fatigue Analysis

The fatigue life of a structure is calculated to predict the length of time to failure when that structure is subject to dynamic motions. The standard method used to calculate fatigue lives of risers uses S-N curves, which relate the number of cycles to failure, \( N \), to the fatigue stress range, \( \sigma_F \), is described by Barltrop & Adams (1991) and takes the form:

\[
N = k \sigma_F^{-m} \tag{8.7}
\]

where

- \( k, m \) are experimentally obtained constants dependant on the connection or weld type and finish. Values for these constants can be found in DNV (1984), DNV (2001) and API (1994).
- \( \sigma_F \) is the fatigue stress range, which is a factored axial stress range that takes the form:
\[ \sigma_F = SCF \sigma_A \] \hspace{1cm} (8.8)

where

- \text{SCF} \quad \text{is a stress concentration factor based on the geometric and finish properties of the riser pipe connections}

Stress fluctuations from many cycles can be summed together using Miner’s rule to calculate the total fatigue damage, where values equal to or greater than 1.0 indicate failure. The formula for maximum fatigue damage is given below.

\[ D = \sum_{n=1}^{N} \frac{1}{N_n} \] \hspace{1cm} (8.9)

where

- \text{D} \quad \text{is the fatigue damage},
- \text{N}_n \quad \text{is the number of cycles to failure for a given stress range},
- \text{N}_\sigma \quad \text{is the number of stress ranges considered}.

Other fatigue parameters used in industry are the fatigue life and the fatigue damage rate. These can be calculated from the fatigue damage by taking account of the length of time of the stress history considered. The damage rate of a stress history is the total fatigue damage of the stress history divided by the length of time of the stress history, as shown below.

\[ D_R = \frac{D}{t_T} = \frac{1}{t_T} \sum_{n=0}^{t_T} \frac{1}{N_n(t)} \] \hspace{1cm} (8.10)

where

- \text{D}_R \quad \text{is the fatigue damage rate},
- \text{t}_T \quad \text{is the total time of the stress history},
- n(t) \quad \text{is the stress range number as a function of time}.

The fatigue life is the length of time require for the sample to fail, and is calculated from the inverse of the fatigue damage rate as shown below:

\[ L = \frac{1}{D_R} \] \hspace{1cm} (8.11)
where

\[ L \] is the fatigue life. This is generally quoted in years by dividing through by the number of seconds in a year, viz: \( \frac{1}{365 \times 24 \times 60 \times 60} \), provided that \( t_1 \) is given in seconds.

For regular sinusoidal motions the fatigue life can be simplified to the following:

\[ L = N t = \frac{k t}{\sqrt[3]{\sigma_F^m}} \]  

(8.12)

where

\[ t \] is the period of the motion.

This shows that fatigue life is inversely proportional to fatigue stress range and proportional to the period of the stress fluctuations.

To compare the relative fatigue damage between two stress ranges the above equations can be rearranged and simplified to form an equation for a damage factor ratio. Examination of S-N curves shows that the empirical fatigue exponent, \( m \), can be generalised using a value of 3.

\[ Df_{R} = \left( \frac{\sigma_{F_a}}{\sigma_{F_b}} \right)^3 \]  

(8.13)

where

\[ Df_{R} \] is a damage factor ratio,

\( \sigma_{F_a}, \sigma_{F_b} \) are the stress ranges being compared.

The stress ranges used in the fatigue damage assessment can be calculated from stress histories generated in the time domain or stress histograms calculated using either time or frequency domain approaches. However because the touchdown point moves, the riser is considered a non-linear structure and time domain approaches are more accurate.
8.3 TDP Modelling

8.3.1 Beam on Elastic Foundations

The bending moment and shear forces on the seabed within the touchdown zone (TDZ) can be estimated using closed form solutions, such as the beam on a semi-infinite elastic foundation using expressions developed by Hetenyi (1946). This solution assumes a semi-infinite beam on an elastic Winkler (1867) seabed (stiffness, or Winkler’s constant, $k$) with moment ($M_0$) and shear forces ($W$) applied to the free end of the beam as shown in Figure 8.1.

![Figure 8.1 – Beam on Semi-infinite Foundation](image)

The equation developed by Hetenyi (1946) to model a beam on an elastic foundation takes the following form:

$$EI \frac{d^4 z}{dx^4} + kz = 0 \quad (8.14)$$

This equation was then solved by Hetenyi (1946) to give solutions for shear force, bending moment, rotation and vertical displacement, from either an applied point load or moment. For a point load of magnitude $W$ the equations are:

$$V_x = -We^{-\lambda x} (\cos \lambda x - \sin \lambda x) \quad (8.15)$$

$$M_x = -\frac{W}{\lambda} e^{-\lambda x} \sin \lambda x \quad (8.16)$$

$$\theta_x = \frac{W}{2EI\lambda^2} e^{-\lambda x} (\cos \lambda x + \sin \lambda x) \quad (8.17)$$

$$z_x = -\frac{W}{2EI\lambda^3} e^{-\lambda x} \cos \lambda x \quad (8.18)$$
And for a bending moment of magnitude \( M_0 \) the equations are:

\[
V_x = -2M_0\lambda e^{-\lambda x} \sin \lambda x
\]  

(8.19)

\[
M_x = M_0 e^{-\lambda x} (\cos \lambda x + \sin \lambda x)
\]  

(8.20)

\[
\theta_x = -\frac{M_0}{EI\lambda} e^{-\lambda x} \cos \lambda x
\]  

(8.21)

\[
z_x = -\frac{M_0}{2EI\lambda^2} e^{-\lambda x} (\sin \lambda x - \cos \lambda x)
\]  

(8.22)

where

- \( V_x \) shear force at a distance \( x \) from the free end,
- \( M_x \) bending moment at a distance \( x \) from the free end,
- \( \theta_x \) rotation at a distance \( x \) from the free end,
- \( z_x \) vertical displacement at a distance \( x \) from the free end,
- \( W \) point load applied at the free end of the beam,
- \( M_0 \) moment applied at the free end of the beam,
- \( \lambda \) constant relating the soil stiffness, \( k \) and the bending stiffness, \( EI \), (dimensions \( L^{-1} \)), given below:

\[
\lambda = \left(\frac{k}{4EI}\right)^{1/4}
\]  

(8.23)

To use the solutions the end forces of the semi-infinite beam on the elastic foundation are required. The bending moment is given by the catenary equations, as shown in Figure 8.2.

\[\text{Catenary Zone} \quad \text{Buried Zone}\]

\[\text{Forces from catenary equations} \quad \text{Equal and opposite forces for equilibrium}\]

\[\text{Figure 8.2 – Interaction Between Catenary and Buried Zones}\]

The axial tension in the system is not considered in this model, but is accounted for in the next section. The vertical force is a restraining force, calculated by assuming
that the vertical displacement at the interface between the catenary and buried zone is zero. This implies that a beam on an elastic foundation models all of the pipe/soil interaction. The vertical force is calculated as shown below:

\[ z_{x-Moment} - z_{x-Point \ Load} = 0 \]  

(8.24)

Expanding equation (6.12) with equations (6.19) and (6.23) and taking \( x=0 \), gives:

\[ \frac{M_{TDP}}{2EI\lambda^2} e^{0} (\sin 0 - \cos 0) - \frac{W}{2EI\lambda^2} e^{0} \cos 0 = 0 \]  

(8.25)

Rearranging for the point load in terms of the applied moment gives:

\[ W = -\lambda M_{TDP} = -\frac{k}{4EI} M_{TDP} \]  

(8.26)

Substituting this result into the beam on an elastic foundation equations generates the following set of equations for modelling the TDZ:

\[ V_x = -\lambda M_{TDP} e^{-\lambda x} (\cos \lambda x + \sin \lambda x) \]  

(8.27)

\[ M_x = M_{TDP} e^{-\lambda x} \cos \lambda x \]  

(8.28)

\[ \theta_x = \frac{M_{TDP}}{2EI\lambda} e^{-\lambda x} (\sin \lambda x - \cos \lambda x) \]  

(8.29)

\[ z_x = -\frac{M_{TDP} e^{-\lambda x}}{2EI\lambda^2} \sin \lambda x \]  

(8.30)

Examples of the shape (penetration depth), shear force and bending moment along the TDZ calculated using the above equations are given in Figure 8.3, Figure 8.4, and Figure 8.5 respectively. These figures use a SCR with a 12.75 inch (0.324m) outer diameter SCR in 1800m water depth with a 12° top angle and a range of seabed stiffness from 1kN/m/m to 10,000kN/m/m. The bending moment at the TDP from the catenary equations is 111kNm. Further details of this example SCR are given in Appendix A.
Example of Penetration Depth Along Seabed from Beam on an Elastic Foundation
12.75in OD, 1800m Water Depth, Spar Vessel, 12° Top Angle
Internal Fluid Oil, Varying Seabed Stiffness

Figure 8.3 – Example of Shape of Riser Along seabed

Example of Shear Force Along Seabed from Beam on an Elastic Foundation
12.75in OD, 1800m Water Depth, Spar Vessel, 12° Top Angle
Internal Fluid Oil, Varying Seabed Stiffness

Figure 8.4 – Example Shear Force Along Seabed
The trends observed in the shear force and bending moment are similar; both start at their maximum values at the catenary zone. These gradually reduce as the distance from the TDP increases. The bending moment reduces to zero at the distance where the shear force reaches a maximum, at distance $x_{M0}$. The bending moment then reaches a maximum positive moment, indicating a reversal in the curvature of the beam, at distance $x_{M\text{-MAX}}$ from the catenary/buried zone interface. After this both bending moments and shear forces reduce to zero. The equations developed for distances $x_{M0}$ and $x_{M\text{-MAX}}$ follow:

$$x_{M0} = \frac{\pi}{2\lambda} \quad (8.34)$$

$$x_{M\text{-MAX}} = \frac{3\pi}{4\lambda} \quad (8.35)$$

The maximum positive bending moment is calculated using the equation derived below and is found to be independent of soil stiffness.

$$M_{POS} = -M_{TDP} \frac{1}{\sqrt{2}} e^{-\frac{3g}{4}} = -0.134 M_{TDP} \quad (8.36)$$

The effect of increasing soil stiffness is to reduce the maximum displacement, reduce the distances $x_{Z\text{-MAX}}$ and $x_{Z0}$ and increase the maximum shear force and change the gradient of the bending moment.

The formulae for a beam on an elastic foundation assume that bending stiffness resists the end loads. They do not account for the effects of stress or geometric stiffness from the axial tension in the riser. For this reason the beam on semi-infinite foundation equations approximately determine the bending moments, shear loads and...
The penetration depth is shaped like a ladle. At the catenary/buried zone interface the penetration depth is zero. As the riser continues into the buried zone the penetration depth increases to a maximum depth, then gradually rises towards the mudline. The TDZ equations developed above are solved to determine the distances from the catenary/buried zone interface to the maximum penetration depth \( x_{Z,MAX} \), and the buried/surface zone interface, \( x_{Z0} \), and the maximum penetration depth, \( z_{MAX} \). A sketch of these distances is given in Figure 8.6 and the equations developed are shown below.

\[
x_{Z,MAX} = \frac{\pi}{4\lambda} \tag{8.31}
\]

\[
x_{Z0} = \frac{\pi}{\lambda} \tag{8.32}
\]

\[
z_{MAX} = M_{TDP} \frac{e^{\frac{-z}{2\sqrt{2EI\lambda^2}}}}{2\sqrt{2EI\lambda^2}} = 0.161 \frac{M_{TDP}}{EI\lambda^2} \tag{8.33}
\]
vertical deflection in the TDZ. A more accurate solution is to include the axial tension in the equations, and this has been done by Hetenyi (1946).

8.3.2 Beam on Elastic Foundation with Axial Tension

The solution for a semi-infinite beam in tension on an elastic foundation was developed by Hetenyi (1946) and is given below. This solution assumes that the beam is the same as the ‘beam on elastic foundation’ solution but with axial tension, \( H \), as shown in Figure 8.7.

\[
E I \frac{d^4 z}{dx^4} - H \frac{dz}{dx} + k z = 0
\]  
\[
(8.37)
\]

This equation has been solved to form the following solutions for shear force, bending moment, rotation and vertical displacement for an applied point load and a moment. For a point load of magnitude \( W \) the equations are:

\[
V_x = -\frac{W}{\beta} \frac{1}{3\alpha^2 - \beta^2} \left[ (3\alpha^2 - \beta^2) \beta \cos \beta x - (3\beta^2 - \alpha^2) \alpha \sin \beta x \right]  
\]  
\[
(8.38)
\]

\[
M_x = -\frac{W}{\beta} \frac{2\alpha^2}{3\alpha^2 - \beta^2} e^{-\alpha x} \sin \beta x  
\]  
\[
(8.39)
\]

\[
\theta_x = -\frac{W}{EI} \frac{1}{3\alpha^2 - \beta^2} \frac{1}{\beta} e^{-\alpha x} (\beta \cos \beta x + \alpha \sin \beta x)  
\]  
\[
(8.40)
\]
\[ z_x = \frac{W}{\beta k} \frac{2\alpha^2}{3\alpha^2 - \beta^2} \frac{e^{-\alpha}}{\beta} \left[ 2\alpha\beta \cos \beta x + \left( \alpha^2 - \beta^2 \right) \sin \beta x \right] \quad (8.41) \]

And for a moment of magnitude \( M_0 \) the equations are:

\[ V_x = -M_o \frac{1}{3\alpha^2 - \beta^2} \frac{1}{\beta} e^{-\alpha} \left[ -4\left( \alpha^2 - \beta^2 \right) \alpha\beta \cos \beta x + \left( \alpha^4 - 6\alpha^2 \beta^2 + \beta^4 \right) \sin \beta x \right] \quad (8.42) \]

\[ M_x = -M_o \frac{1}{3\alpha^2 - \beta^2} \frac{1}{\beta} e^{-\alpha} \left[ \left( 3\alpha^2 - \beta^2 \right) \beta \cos \beta x - \left( \alpha^2 - 3\beta^2 \right) \alpha \sin \beta x \right] \quad (8.43) \]

\[ \theta_x = -\frac{M_o}{EI} \frac{1}{3\alpha^2 - \beta^2} \frac{1}{\beta} e^{-\alpha} \left[ 2\alpha\beta \cos \beta x - \left( \alpha^2 - \beta^2 \right) \sin \beta x \right] \quad (8.44) \]

\[ z_x = \frac{M_o}{EI} \frac{1}{3\alpha^2 - \beta^2} \frac{1}{\beta} e^{-\alpha} \left( \beta \cos \beta x - \alpha \sin \beta x \right) \quad (8.45) \]

where \( \alpha \) and \( \beta \) are chosen depending on whether tension stiffness or bending stiffness dominate the response in the TDZ, as shown below.

For \( H < 2\sqrt{kEI} \)

\[ \alpha = \sqrt{\lambda^2 + \frac{H}{4EI}} \quad \beta = \frac{\lambda^2 - \frac{H}{4EI}}{\sqrt{\lambda^2}} \]

For \( H = 2\sqrt{kEI} \)

\[ \alpha = \frac{H}{4EI} \quad \beta = 0 \]

For \( H > 2\sqrt{kEI} \)

\[ \alpha = \sqrt{\lambda^2 + \frac{H}{4EI}} \quad \beta = \frac{H}{4EI} - \lambda^2 \]

A relationship between the TDP point load and TDP bending moment is determined using the same assumption as for the beam on an elastic foundation; that the displacement due to the moment is negated with a shear load. This gives:

\[ W = -\lambda M_{TDP} \frac{\lambda}{\alpha} \quad (8.46) \]

Examples of the shape, shear force and bending moment are given in Figure 8.8 using the example SCR given previously with a seabed stiffness of 100kN/m/m. Figure 8.8 shows that the trends of the shape and force curves are comparable with the beam on an elastic foundation, however the distances to the maximum penetration depth and the buried/surface zone interface are lower. Using the tension beam TDZ solution the distances \( x_{e\text{\,MAX}} \) and \( x_{e0} \) can be derived, and are as follows.
\[
Z_{MAX} = \frac{1}{\beta} \arctan\left(\frac{\beta}{\alpha}\right) \tag{8.47}
\]

\[
Z_0 = \frac{\pi}{\beta} \tag{8.48}
\]

Figure 8.8 — SCR Profile using Tension beam on Winkler Springs, 
\( k = 100\text{kN/m/m} \)
Examination of the equations shows that if the seabed stiffness or bending stiffness is high compared to the TDP tension then the tension beam on the elastic foundation TDZ model is approximately equal to the beam on elastic foundation TDZ model. This implies that, although the tension beam on an elastic foundation is a more accurate model of a SCR, for the simplicity of the equations, the beam on elastic foundation equations can be used to examine TDZ pipe/soil interaction.

8.4 Discussion
8.4.1 Maximum Stress

The beam on elastic foundation TDZ model equations show that the maximum bending stress at the TDP of a SCR is determined by the geometric properties of the riser in the catenary zone and not by the soil stiffness. This result also implies that properties of the trench model will have little impact on the maximum TDP stress.

8.4.2 Fatigue Analysis

The stress ranges used in the fatigue analysis of SCRs are calculated from the changes in riser stress caused by first and second order motions. For the TDZ model these motions can be simplified to moving the TDP in-line with the riser and the stress ranges dominated by bending moment. A sketch of the change in the bending moments in the TDZ due to example riser motions with both stiff and soft soil stiffnesses is shown in Figure 8.9.

![Figure 8.9 - Stress Used in Fatigue Calculations](image_url)
The sketch of the bending moments in the TDZ, Figure 8.9, shows that the range of the fatigue stress in the TDZ is dependent on the rate of change of the bending moment. From mechanics equations the gradient of the bending moment is equal to the shear force. The beam on elastic foundation equations show that the maximum shear force is proportional to the soil stiffness, indicating that riser fatigue damage is also proportional to soil stiffness.

The increase in fatigue damage between two soil stiffnesses can be calculated using the damage factor ratio described previously.

\[
Df_R = \left( \frac{\sigma_{AXIAL,k1}}{\sigma_{AXIAL,k2}} \right)^3 = \left( \frac{W_{k1}}{W_{k2}} \right)
\]  

(8.49)

The maximum shear force, \(W\), is substituted by \(\lambda M_{TDP}\), as shown below.

\[
Df_R = \left( \frac{-\lambda_{k1}M_{TDP}}{-\lambda_{k2}M_{TDP}} \right)^3
\]

(8.50)

And this simplifies to the following equation for damage factor ratio between two different linear soil stiffnesses.

\[
Df_R = \left( \frac{k_1}{k_2} \right)^3
\]

(8.51)

This equation shows that as the soil stiffness increases by an order of magnitude, the fatigue damage in the SCR increases by 562%. This indicates that accurate pipe/soil interaction models and measurements of soil properties are essential for SCR analysis.

8.5 Summary and Conclusions

Two closed-form TDZ models have been developed in this section. They use the beam on elastic foundation and tension beam on elastic foundation solutions developed by Hetenyi (1946) for Winkler foundations. The end forces used in the models are determined using the catenary equations that are detailed in Appendix A. The beam on an elastic foundation provides a series of relatively straightforward equations that can be manipulated to show the effect of soil stiffness on penetration
depth, trench shape, shear force and bending moment along the TDZ. The tension beam on an elastic foundation TDZ model is more accurate, but also more complex to manipulate.

Both models use linear Winkler foundations. An improved model would use a non-linear Winkler foundation, such as described by Brandenburg & Boulanger (2004). However this approach would require the use of finite element solutions which are not considered in this section of the thesis.

The TDZ models are shown to describe the initial trench shape and the shear force and bending moment distributions along the buried zone. By manipulation of the equations of the TDZ model distances that describe the trench shape have been found. These can be used in SCR analysis to estimate the maximum initial trench penetration and the length of the trench.

The TDZ models have been used to show that the maximum TDP stress is independent of the soil stiffness. However the models also show that riser fatigue damage is dependent on the soil stiffness, and that for each order of magnitude increase in soil stiffness the fatigue damage increases by 562%.

The models developed in this section are similar to those published by Hahn et al (2003) where modified catenary equations were combined with Hetenyi’s (1946) tension beam on an elastic foundation. Hahn et al (2003) also showed that the riser fatigue life was dependant on the value of soil stiffness used, but did not, however, develop a relationship to describe the increase.
9.0 ANALYTICAL SCR MODELLING

9.1 Introduction

Detailed analysis of all riser systems is generally conducted using finite element analysis (FEA). Consequently the pipe/soil interaction models developed in the previous chapters need to be used in FEA codes. This sections outlines a method used to construct steel catenary risers (SCRs) within FEA programs and the method used to incorporate the pipe/soil suction model into the ANSYS FEA code. Analysis is conducted using an example full scale SCR model in 1800m water depth in a Gulf of Mexico environment to confirm the beam on an elastic foundation model for touchdown point (TDP) interaction and to show the effect of seabed stiffness, seabed slope and pipe/soil suction during static, slow drift, extreme storm and fatigue loading conditions.

9.2 Finite Element Analysis

SCRs are designed using numerical models that determine the riser response to static and dynamic loads. They can be modelled using closed form solutions, as shown in the previous chapter, but are generally analysed using FEA due to their complex and dynamic nature. Typically risers are modelled globally using line elements that have the capability to model system mass, bending and axial stiffness, top and bottom boundary conditions, such as the vessel and the seabed, and hydrodynamic loads, as detailed by 2H Offshore (2002b). There are many finite element (FE) codes available that can model these factors and are used for riser analysis. However the FE codes available for this study are FLEXCOM (MCS, 2004) a specialist riser analysis code and ANSYS (ANSYS Inc, 2000) a general-purpose FE code. A summary of these FE codes is given in chapter 2, section 2.3.2.

9.3 Modelling SCRs in ANSYS

9.3.1 Modelling a Catenary

To model a SCR in a FE code the catenary shape has to be constructed from a straight pipe to correctly model the bending moment and stress distribution along the riser. If the un-deformed shape of the riser is modelled as a catenary the bending
moment distribution along the riser will be zero. There are a number of ways to construct the catenary shape, one is to use a catenary solver that automates the process, altering the local element axes so they are horizontal. A second is to use the method presented below which forms a straight pipe into a catenary shape. The catenary solver is available in specialist riser codes, such as FLEXCOM by MCS (2004) but not in general purpose codes such as ANSYS by ANSYS Ltd (2000). This does not, however, exclude the possibility that one day catenary solvers may exist in general purpose FE codes.

The method for modelling a static steel catenary riser in a standard FE package without a catenary solver is described below. The modelling description assumes that the axis taken follow the right hand rule with Z as the vertical axis, X as the horizontal axis and Y going away from the page. The catenary riser is initially modelled as a straight pipe on the seabed as shown in Figure 9.1, picture A. Both ends of the pipe are supported in Y and Z planes, while only the tail end of the pipe is fixed in the X plane. The end of the pipe which is to be attached to the vessel has a large restraining force applied which acts as the support. Note that at the current load step gravity is assumed to be zero.

The end of the pipe that is to be attached to the vessel is then lifted to the vessel attachment height as shown in Figure 9.1 picture B. The horizontal force acts as a lateral restraint, but allows the top of the riser to move so as not to over stress the pipe and cause iteration problems. After this step the vessel end of the riser is fixed in the X, Y and Z axes and the horizontal force removed.

The next load step applies gravity to the model. A catenary curve develops along the riser due to riser self-weight. This is shown in Figure 9.1 picture C. The vessel end of the riser is now moved in the X direction towards the tail end of the riser into the nominal vessel location. Since the final vessel position may be dependant of the riser/vessel connection angle fine-tuning is achieved by moving the riser in the X plane.
Within the ANSYS FE code the element chosen for the riser model is the PIPE59 element that is described in detail by ANSYS (2000). This element is a uniaxial element with tension-compression, torsion, and bending capabilities, and with member forces simulating ocean waves and current. The element has six degrees of freedom at each node: translations in the nodal x, y, and z directions and rotations about the nodal x, y, and z axes has the capability to model hydrodynamic and buoyant effects of the water. The element mass includes the added mass of the water and the effects of pipe internal fluids.

### 9.3.2 Pipe/Soil Suction Modelling

The pipe/soil suction interaction model has been developed in the ANSYS FE code using the ANSYS parametric design language (APDL), which allows standard
verified building blocks, such as springs and damping elements, to be combined into complex models. The pipe/soil suction model contains a non-linear spring element (ANSYS element name COMBIN39), representing the soil, coupled in series to a directional control element (ANSYS element name COMBIN37) that allows the pipe to breakout from the soil. This is illustrated in Figure 9.2 and described in detail below.

The pipe/soil suction curve is modelled using the non-linear spring. This is set up so that the positive, or tension force models the pipe/soil suction curve and compression forces represent pipe/soil stiffness. When the model is initialised the directional control element is axially stiff, or ON, so that any pipe/soil interaction is handled by the non-linear spring. The directional control element remains axially stiff until the displacement in the tension direction reaches the break-out displacement, at which point the axial stiffness reduces to near 0Nm² and the control element is said to be off. This effectively decouples the non-linear spring from the riser. The directional control element remains off until the displacement reaches zero, at which point the directional control element turns on again, which re-couples the non-linear spring to the pipe. This is illustrated in Figure 9.3. If, during the pull-out, the pipe displacement is less than the break-out displacement then when the pipe is pushed back into the soil, the directional control element remains on and the model follows the pipe/soil suction curve. This is illustrated in Figure 9.4.

Figure 9.2 – Pipe/Soil Interaction Model
9.4 Design Basis for Example SCR

9.4.1 Riser Arrangement

The SCR used in the analysis was created during the STRIDE JIP by 2H Offshore (2002d) and is representative of deepwater SCRs in the Gulf of Mexico. A summary of the global properties of the riser is given in Table 9.1 and a sketch in Figure 9.5.
The cross-sectional pipe properties of the SCR are summarised in Table 9.2 and properties typical of deepwater Gulf of Mexico sediments are given in Table 9.3. The wave data used in the analysis is given in Table 9.4 and generic Spar response amplitude operators (RAOs) given in Figure 9.6.

### Table 9.1 – Global SCR Properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Depth</td>
<td>1800m</td>
</tr>
<tr>
<td>Height of Riser Attachment Point, $z_A$</td>
<td>1600m</td>
</tr>
<tr>
<td>Nominal Top Angle, $\theta$</td>
<td>12°</td>
</tr>
<tr>
<td>Vessel Type</td>
<td>Spar</td>
</tr>
</tbody>
</table>

![Figure 9.5 - Gulf of Mexico Spar SCR General Arrangement](image-url)
### Table 9.2 – Riser Pipe Properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outer Diameter, $D_0$</td>
<td>0.324m (12%&quot;&quot;)</td>
</tr>
<tr>
<td>Wall Thickness, $t$</td>
<td>0.0205m</td>
</tr>
<tr>
<td>Cross-sectional Area of Steel, $A_s$</td>
<td>0.0195m$^2$</td>
</tr>
<tr>
<td>Second Moment of Area, $I$</td>
<td>2.26x$10^{-4}$ m$^4$</td>
</tr>
<tr>
<td>Coating Thickness, $t_E$</td>
<td>50mm (2.0&quot;)</td>
</tr>
<tr>
<td>Steel Density, $\rho_S$</td>
<td>7850 kg/m$^3$</td>
</tr>
<tr>
<td>External Coating Density, $\rho_E$</td>
<td>700 kg/m$^3$</td>
</tr>
<tr>
<td>Internal Fluid Density, $\rho_I$</td>
<td>800 kg/m$^3$ (Oil)</td>
</tr>
<tr>
<td>In Service Weight in Water, $m_S$</td>
<td>100 kg/m</td>
</tr>
<tr>
<td>Bending Stiffness, $E I$</td>
<td>4.67x$10^7$ Nm$^2$</td>
</tr>
<tr>
<td>Axial Stiffness, $E A_s$</td>
<td>4.04x$10^9$ N</td>
</tr>
<tr>
<td>Fatigue Curve</td>
<td>DNV (1984) E Class Weld</td>
</tr>
<tr>
<td></td>
<td>$k = 1.05x10^{12}$, $m = 3.0$, SCF = 1.1</td>
</tr>
</tbody>
</table>

### Table 9.3 – Soil Properties for Gulf of Mexico

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface Undrained Shear Strength</td>
<td>2.63 kPa</td>
</tr>
<tr>
<td>Undrained Shear Strength Gradient</td>
<td>1.26 kPa/m</td>
</tr>
<tr>
<td>Submerged Unit Weight of Soil</td>
<td>4.4 kN/m$^3$</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>50%</td>
</tr>
<tr>
<td>Sensitivity of the Clay</td>
<td>3.0</td>
</tr>
<tr>
<td>Assumed Trench Depth</td>
<td>4.0D (1.3m)</td>
</tr>
</tbody>
</table>
Table 9.4 – Environmental Data

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>100yr Hurricane Wave Height</td>
<td>10.7m</td>
</tr>
<tr>
<td>100yr Hurricane Wave Period</td>
<td>13.0s</td>
</tr>
<tr>
<td>Random Sea Wave Height</td>
<td>2.5m</td>
</tr>
<tr>
<td>Random Sea Wave Period</td>
<td>7.0 – 9.0s</td>
</tr>
<tr>
<td>Random Sea Gamma Value</td>
<td>0.0119548</td>
</tr>
</tbody>
</table>

Figure 9.6 – Generic Truss Spar RAOs

9.4.2 Soil Stiffness

The values of vertical pipe/soil stiffness used in this study are summarised in Table 9.5 and were based on those calculated by the pipe/soil interaction models presented in Chapter 7. An indication of the pipe/soil interaction model associated with the soil stiffnesses is also given.
Table 9.5 – Seabed Stiffness for Example SCR

<table>
<thead>
<tr>
<th>Soil Stiffness Values are Typical of X Models</th>
<th>Seabed Type</th>
<th>Soil Stiffness (kN/m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static Pipe/Soil Interaction Models</td>
<td>Extremely Soft</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Very Soft</td>
<td>10</td>
</tr>
<tr>
<td>Small and Large Displacement Pipe/Soil</td>
<td>Soft</td>
<td>430</td>
</tr>
<tr>
<td>Interaction Models</td>
<td>Firm</td>
<td>1,200</td>
</tr>
<tr>
<td>Cyclic Pipe/Soil Interaction Models</td>
<td>Stiff</td>
<td>5,000</td>
</tr>
<tr>
<td>Conservative Analysis</td>
<td>Rigid</td>
<td>- N/A -</td>
</tr>
</tbody>
</table>

9.4.3 Seabed Slope

Based on the observation and conclusions from the ROV trench surveys it is assumed that a SCR trench within the dynamic region can be modelled as a sloping seabed. Within the analysis the seabed slope is modelled using a rigid seabed with uniform gradient of ±5%, ±3%, ±1% and 0% where a positive gradient is defined as a slope that has greater depth at the TDP than the well as shown in Figure 9.7. For consistency the seabed rotation is always carried out about the SCR TDP

---

Figure 9.7 – Definition of Seabed Slope
9.4.4 Pipe/Soil Suction Model

The pipe/soil suction curves for \( \frac{1}{2}D \), 2D, 3D and 4D trench depths are given in Figure 9.8. Two sets of curves are presented relating to pull-out from vessel drift and dynamic cyclic loading from wave action for extreme storm and fatigue analysis. The maximum pipe/soil suction forces per unit length and break out displacements for both drift and cyclic loading are summarised in Table 9.6. The maximum pipe/soil suction forces for the dynamic curves are of the order of 0.1 times those for the drift curves while the break out displacements are of the order of one-third of the values for the drift curves.

<table>
<thead>
<tr>
<th>Trench Depth</th>
<th>Maximum Pipe/Soil Suction Force (kN/m)</th>
<th>Break-out Displacement (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Drift</td>
<td>Cyclic</td>
</tr>
<tr>
<td>( \frac{1}{2}D )</td>
<td>7.45</td>
<td>0.65</td>
</tr>
<tr>
<td>2D</td>
<td>9.51</td>
<td>0.83</td>
</tr>
<tr>
<td>4D</td>
<td>12.26</td>
<td>1.07</td>
</tr>
<tr>
<td>6D</td>
<td>15.00</td>
<td>1.31</td>
</tr>
</tbody>
</table>
9.5 Analysis Conducted

Analysis is conducted to determine the effect of soil stiffness, seabed slope and pipe/soil suction on SCR maximum TDP stress, stress distribution along the TDZ and fatigue life. An overview of the types of analysis conducted is given below:

- **Static analysis** – The riser is analysed statically under self weight and buoyancy loads. Comparisons are made of the SCR profile and bending moment.
- **Slow drift** – The riser is analysed dynamically under self weight and buoyancy loads for motions of the vessel from the 5% of water depth near position to nominal, and from nominal to 5% of water depth far position. Comparisons are made of maximum von Mises stress.
- **Extreme wave** – The riser is analysed dynamically under self weight, buoyancy loads and wave loads arising from a 100-year hurricane. The analysis is conducted at ±5% of water depth vessel offsets. Comparisons are made of maximum von Mises stress.
- **First order fatigue analysis** – The riser are subject to time-domain random wave analysis. Comparisons are made of the maximum fatigue damage.
The original SCR seabed model consists of a rigid flat seabed with no pipe/soil suction capabilities. In each of the analyses conducted the effects of soil stiffness, seabed slope and pipe/soil suction were assessed independently of each other.

9.6 Static Analysis

9.6.1 Overview

The objectives of this analysis were to determine the effects of soil stiffness and seabed slope on static SCR response. The riser was analysed statically under self weight and buoyancy loads. The soil stiffness and seabed slope are varied independently of each other. Analysis is conducted at 5% of water depth near, nominal and 5% water depth far vessel offsets. Comparisons are made of SCR profile, bending moment and von Mises stress.

9.6.2 Soil Stiffness

The effect of pipe/soil stiffness on SCR profile is shown, with a close up of the TDZ, in Figure 9.9. The bending moment along the riser within the TDZ is shown in Figure 9.10. From these plots the following observations are made:

- There is no change in the global shape of the riser with changes in soil stiffness.
- It can be seen that as the soil stiffness reduces the static pipe embedment increases, and the TDP moves towards the vessel.
- The maximum bending moment remains the same for all soil stiffnesses.
- At the TDP the change in bending moment with riser length is abrupt for the rigid seabed and high soil stiffnesses. For lower soil stiffness the change in bending moment with riser length is more gradual.
9.6.3 Seabed Slope

The effect of seabed slope on static SCR configuration is shown in Figure 9.11 with a close up of the TDZ in Figure 9.12. The bending moment distribution along the
riser length is shown in Figure 9.13 with a close up of the sagging bending moment peaks in Figure 9.14. The following observations are made:

- A positive seabed slope moves the TDP away from the vessel by approximately 20m for every 5% of water depth change in vessel position.
- A 5% seabed slope increases the maximum bending moment by about 0.9%, while a −5% seabed slope reduces the maximum bending moment by 1.4%.

![Static Side Elevation of the SCR with the Seabed Profile at TDP](image)

Figure 9.11 – Static Configuration of Riser with a Range of Seabed Slopes
Figure 9.12 – Static Configuration: Close Up of TDP

Figure 9.13 – Bending Moment Distribution Along SCR with a Range of Seabed Slopes
9.7 SCR Response to Slow Drift Motions

9.7.1 Overview

The objectives of this analysis are to assess the effect of pipe/soil suction during riser pull-out on von Mises stress. The analysis approach is to move the vessel from the 5% water depth near offset to the nominal, and as a separate analysis move the vessel from the nominal offset to the 5% water depth far offset. These motions pull the riser away from the seabed causing the TDP to move towards the well and mobilising the pipe/soil suction force. The riser is analysed quasi-statically with and without the pipe/soil suction model.

9.7.2 Pipe/Soil Suction

The effects of pipe/soil suction on maximum von Mises stress due to slow drift vessel motions are shown in Figure 9.15 and Figure 9.16 for 5% water depth vessel near offset to nominal and nominal to 5% water depth vessel far offset respectively. The percentage increase in von Mises stress for each vessel slow drift motion is summarised in Figure 9.17. These figures show that the maximum von Mises stress in the riser increases by approximately 50% due to the pipe/soil suction force in a 6D
trench. In the shallower trench depth where the pipe/soil suction force is weaker the increase in von Mises stress is approximately 28%, 34% and 45% for the ½D, 2D and 4D trench depths respectively.

Figure 9.15 – Maximum von Mises Stress Envelopes for Vessel Motion 5% Near Offset to Nominal

Figure 9.16 – Maximum von Mises Stress Envelopes for Vessel Motion Nominal to 5% Far Offset
9.8 SCR Response to Extreme Storm Analysis

9.8.1 Overview

The objectives of this analysis were to examine the effects of soil stiffness, seabed slope and pipe/soil suction on a SCR during extreme storm loading. The riser is analysed with the wave and current loading associated with a 100 year hurricane event. For the soil stiffness and the seabed slope studies the riser is analysed in the 5% water depth near, nominal and 5% water depth far vessel offsets, while for the pipe/soil suction analysis the riser is analysed in the nominal vessel position. Comparisons are made of von Mises stress distribution along the riser length and maximum von Mises stress at the TDP.

9.8.2 Soil Stiffness

The distribution of von Mises stress along the SCR in the TDZ with different levels of soil stiffness with the vessel in the nominal position is shown in Figure 9.18. The results show that the maximum von Mises stress due to extreme storm loading does not change with soil stiffness. However the shape of the von Mises stress distribution

Figure 9.17 – Percentage Change in Maximum von Mises Stress due to Pipe/Soil Suction with Trench Depth
does change with soil stiffness; for low soil stiffness the slope of the von Mises stress distribution is gradual, where as for the rigid seabed the slope is abrupt.

A summary of the maximum von Mises stress along the riser for the 5% water depth near, nominal and 5% water depth far vessel offsets is given in Figure 9.19 and shows that for each vessel offset considered there is no change in the maximum von Mises stress with soil stiffness.

Figure 9.18 – Maximum von Mises Stress Along TDZ due to Extreme Wave with Different Seabed Stiffnesses
Figure 9.19 – Summary of Maximum von Mises Stress due to Extreme Storm Wave with Varying Seabed Stiffness

9.8.3 Seabed Slope

The von Mises stress distribution along the riser length due to extreme storm loading at the nominal vessel position with seabed slopes between ±5% gradient are shown in Figure 9.20. A summary of the maximum von Mises stress for the riser analysed in the 5% water depth near, nominal and 5% water depth far vessel offsets is given in Figure 9.21. The graphs show that the maximum von Mises stress and the stress distribution along the riser length does not change significantly with seabed slope.
Figure 9.20 – von Mises Stress Distribution Along TDZ at Nominal Vessel Offset due to Extreme Storm Loads for a Range of Seabed Slopes

Figure 9.21 – Maximum von Mises Stress at 5% Near, Nominal and Far Vessel Offsets due to Extreme Storm Loads for a Range of Seabed Slopes
9.8.4 Pipe/Soil Suction

The effects of pipe/soil suction on maximum von Mises stress during extreme storm (100 year hurricane wave) loading at vessel offsets of 0% and ±5% water depth are shown in Figure 9.22 and summarised in Figure 9.23. The figures show that the pipe/soil suction force increases the von Mises stress at the TDP by a maximum of 0.7%, which is a small change in stress.

![Figure 9.22 – Maximum von Mises Stress to Yield Stress Ratio for 100 year Hurricane Wave](image-url)
9.9  First Order Fatigue Analysis

9.9.1  Overview

The objectives of this analysis are to assess the effects of soil stiffness, seabed slope and pipe/soil suction on first order fatigue damage. The analysis was conducted with the riser in the nominal position and a time domain random sea of a single sea state applied to the vessel. The stress timetraces around the SCR within the TDZ were calculated and a rainflow cycle counting algorithm used to calculate the fatigue damage.

9.9.2  Soil Stiffness

The distribution of fatigue damage along the TDZ using soil stiffnesses from 1kN/m/m to 10,000kN/m/m and a rigid seabed are shown in Figure 9.24 and given in Table 9.7. A summary of the maximum fatigue damage as a percent of the maximum rigid seabed damage is given in Figure 9.25. These show that the maximum fatigue damage on the rigid seabed and 10,000kN/m/m stiffness seabed occurs at 1984m from the vessel end of the riser and that the location of the maximum fatigue damage moves towards the vessel as the seabed stiffness reduces so that with the 1kN/m/m...
seabed the TDP is at 1971m from the vessel. It can be seen that as the soil stiffness reduces from 10,000kN/m/m to 1kN/m/m that the maximum fatigue damage reduces to 2.3%, from 0.0178 1/years to 0.0004 1/years, indicating that the fatigue life increases by 4200% from 56years to 2369years. The shape of the fatigue distribution also changes so that with high soil stiffnesses and the rigid seabed the fatigue damage distribution shape is peaked and concentrated along a small distance of riser. As the soil stiffness is reduced the fatigue damage distribution spreads along larger proportion of the TDZ.

Table 9.7 – Summary of Maximum Fatigue Damages and Lives for Changing Soil Stiffness

<table>
<thead>
<tr>
<th>Soil Stiffness (kN/m/m)</th>
<th>Fatigue Damage (1/years)</th>
<th>Fatigue Life (Years)</th>
<th>% of Rigid Seabed Fatigue Damage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0004</td>
<td>2369</td>
<td>2.4</td>
</tr>
<tr>
<td>4</td>
<td>0.0007</td>
<td>1427</td>
<td>3.9</td>
</tr>
<tr>
<td>10</td>
<td>0.0015</td>
<td>668</td>
<td>8.4</td>
</tr>
<tr>
<td>110</td>
<td>0.0058</td>
<td>173</td>
<td>32.5</td>
</tr>
<tr>
<td>430</td>
<td>0.0088</td>
<td>114</td>
<td>49.4</td>
</tr>
<tr>
<td>1,200</td>
<td>0.0123</td>
<td>81.6</td>
<td>68.8</td>
</tr>
<tr>
<td>1,900</td>
<td>0.0133</td>
<td>75.1</td>
<td>74.7</td>
</tr>
<tr>
<td>5,000</td>
<td>0.0172</td>
<td>58.0</td>
<td>96.7</td>
</tr>
<tr>
<td>10,000</td>
<td>0.0178</td>
<td>56.1</td>
<td>100.0</td>
</tr>
<tr>
<td>Rigid Seabed</td>
<td>0.0178</td>
<td>56.1</td>
<td>100.0</td>
</tr>
</tbody>
</table>
Figures 9.24 and 9.25 illustrate the effects of varying seabed stiffness on the fatigue life distribution and the percentage of rigid seabed fatigue damage of the SCR. The distribution of fatigue damage along the TDZ with changing seabed slopes from -3%, -1%, flat, 1% and 3% gradients is shown in Figure 9.26. The maximum fatigue...
damages and fatigue lives are summarised in Table 9.8. The percent change in the maximum fatigue damage with seabed slope is shown in Figure 9.27. From these figures the following observations are made:

- As the seabed gradient increase the maximum fatigue damage moves towards the vessel
- As the gradient of the seabed increases to 1% the maximum fatigue damage decreases to 5.8% of a flat ridged seabed. As the gradient continues to increase the fatigue damage remains unchanging, at 5.7% of the flat ridged seabed fatigue damage
- As the gradient of the seabed decreases to -3% the maximum fatigue damage increases to 4.2% greater than that of a rigid seabed.

### Table 9.8 – Summary of Maximum Fatigue Damages and Lives for Changing Seabed Slope

<table>
<thead>
<tr>
<th>Soil Slope (%)</th>
<th>Fatigue Damage (1/years)</th>
<th>Fatigue Life (Years)</th>
<th>% of Rigid Seabed Fatigue Damage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-3</td>
<td>0.0186</td>
<td>53.8</td>
<td>4.2</td>
</tr>
<tr>
<td>-1</td>
<td>0.0180</td>
<td>55.5</td>
<td>1.1</td>
</tr>
<tr>
<td>0</td>
<td>0.0178</td>
<td>56.1</td>
<td>0.0</td>
</tr>
<tr>
<td>1</td>
<td>0.0168</td>
<td>59.6</td>
<td>-5.8</td>
</tr>
<tr>
<td>3</td>
<td>0.0168</td>
<td>59.5</td>
<td>-5.7</td>
</tr>
</tbody>
</table>
9.9.4 Pipe/Soil Suction

The effect of pipe/soil suction on the fatigue life along the riser is shown in Figure 9.28. The minimum fatigue lives calculated are given in Table 9.9. These show that
the fatigue damage with and without the pipe/soil suction force are similar as including the pipe/soil suction force in the analysis changes the maximum fatigue damage by less than 1%. It is also observed that the location of the minimum fatigue damage has moved towards the vessel by approximately 1.5m.

Table 9.9 – Effects of Pipe/Soil Suction on Minimum Fatigue Lives

<table>
<thead>
<tr>
<th>Analysis</th>
<th>Maximum Fatigue Damage (1/Years)</th>
<th>Minimum Fatigue Life (Years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Pipe/soil Suction</td>
<td>0.0100</td>
<td>99.6</td>
</tr>
<tr>
<td>Pipe/soil Suction</td>
<td>0.0099</td>
<td>101</td>
</tr>
<tr>
<td>% Difference</td>
<td>-0.89%</td>
<td>+0.89%</td>
</tr>
</tbody>
</table>

![SCR Fatigue Damage With and Without Pipe/Soil Suction](image)

Figure 9.28 – Fatigue Life Versus Location Along the Riser

9.10 Discussion and Conclusions

9.10.1 Overview

This section has detailed the analysis of a SCR with 12¾-inch outer diameter in 1800m water depth connected to a spar vessel in the Gulf of Mexico. Analysis has
been conducted to assess the effects of soil stiffness, seabed slope (representing the SCR trench) and pipe/soil suction on this SCR while under static, slow drift, extreme storm and fatigue analysis loading. It is important to note that the results presented are applicable to this SCR and may be different to other configurations of top angle, water depth and riser diameter. Sensitivity analysis to examine the effects of soil stiffness, seabed slope and pipe/soil suction should always be conducted during SCR design.

9.10.2 Modelling SCRs

The first sections of the chapter describe a method for creating a catenary riser in general purpose FE codes where catenary solvers do not exist. The method for modelling the pipe/soil suction model within the ANSYS FE code is also developed and shown. This uses a non-linear spring and control element so that dynamic pipe/soil suction analysis can be conducted.

9.10.3 Soil Stiffness

The effect of soil stiffness on SCR profile, stress distribution and fatigue damage was assessed using nine soil stiffnesses that varied from very soft (1kN/m/m) to very stiff (10,000kN/m/m) and used a rigid flat seabed as a control. The analysis shows that, as expected, decreasing the soil stiffness increases the penetration depth and moves the TDP towards the vessel. It is also observed that a soil stiffness of 10,000kN/m/m has consistently similar results to those of the rigid seabed. Under extreme storm loads the analysis shows that the value of soil stiffness used does not alter the maximum stress but does change the shape of the stress distribution along the riser, for the rigid and stiff seabeds the change in stress is abrupt, while for soft seabeds the change in stress is gradual. The change in the stress distribution is not considered significant for extreme storm analysis as generally the maximum stress is important as it is used to calculate riser wall thickness and operating windows. However the change in the stress distribution does effect the first order fatigue results as it is the change in dynamic stress along the TDZ that determines the stress used to calculate the fatigue damage. Here it can be seen that the rigid and stiff seabeds have high fatigue
damages while soft seabeds have low fatigue damages, a seabed stiffness of 110kN/m/m has approximately three times more fatigue life than a stiff seabed.

The FEA conducted agrees with the closed form model developed in the previous chapter. The implications of this are that, for simplicity, static, slow drift and extreme storm analysis can be conducted using rigid surfaces as the maximum stress are the same as those analysis conducted on elastic seabeds. However for first order fatigue analysis where the seabed is represented using a rigid seabed the fatigue damage can be considered conservative. The analysis also shows less conservative and hence more realistic fatigue lives can be obtained using elastic seabed, but the value of soil stiffness must be calculated using the appropriate dynamic pipe/soil interaction model and assumptions, else the riser fatigue life will be unrealistically high.

9.10.4 Seabed Slope

The effect of the seabed slope, representing the dynamic portion of a SCR trench, on a SCR is conducted. The results presented used a seabed slope gradient that varied between ±5%. The static and extreme storm analyses show that as the seabed slope changes the SCR profile also changes which results in a change in the maximum bending moment at the TDP of under 1.5%. This change in stress is considered small and not significant for riser design where a 2% error in stress can easily occur due to FE modelling assumptions. The effect of seabed slope on the first order fatigue analysis was also shown to be small, as the change in fatigue damage at a gradient of 3% was less 6%. Generally a safety factor of ten is applied to the results from fatigue analysis, indicating that the effects of seabed slope on a SCR design are minimal and can be ignored.

9.10.5 Pipe/Soil Suction

Two sets of pipe/soil suction curves are created for SCR analysis and were based on the pipe/soil suction models developed in previous chapters of this thesis. A static set for the slow drift analysis and a dynamic set, where the pipe/soil suction force is 10% of the static pipe/soil suction force for extreme storm and fatigue analysis. The results of the analysis on the example riser show that pipe/soil suction is significant
during slow drift vessel motions, where the riser is pulled up and away from the seabed. However during extreme storm and first order fatigue analysis where there are small cyclic TDP motions the effect of pipe/soil suction has been shown to be slight; the maximum riser stress was increased by 0.7% and the first order fatigue life reduced by 0.9%.

This indicates that pipe/soil suction should be considered during large TDP motions that could occur during installation, failed mooring line or extreme storm conditions, and can be ignored for fatigue calculations. However sensitivity analysis should always be conducted to show that these conclusions are true.
10.0 DISCUSSION AND CONCLUSIONS

10.1 Introduction

Many factors influence the way a steel catenary riser (SCR) reacts with the soil at the point where it lays down on the seabed. Through evaluation of previous work, processing of test data and model simulation, the key parameters affecting the touchdown point (TDP) response have been identified and, in addition, modelling methods have been proposed for determining response.

Typically, SCR analysis uses either rigid or linear elastic surfaces to model the seabed. This body of work shows that using a rigid surface to model the seabed produces conservative analysis results (under-predicted fatigue lives), whilst the elastic surface may be non-conservative if an incorrect soil stiffness is used, or over-conservative if too stiff. Additionally, neither seabed models explicitly account for the shape of the seabed within the touchdown zone (TDZ) nor the pipe/soil suction forces, both of which can affect both the riser maximum stress and the fatigue life.

The main focus of this thesis has been to evaluate the effects of pipe/seabed interaction on SCRs. This has included examination of SCR trenches to develop a generic trench profile, development of pipe/soil interaction models, and determination of their effect on SCR maximum stress and fatigue life.

10.2 SCR Trenches

The seabed trenches of existing SCRs have been examined using remotely operated vehicle (ROV) survey data. The ROV survey videos has allowed the writer to correlate observations of trench cross-sectional shapes with measurements of trench widths and depths, and develop a generic trench profile that was “ladle” shaped in profile and bell mouth shaped in plan, Figure 3.22. Many trench features have also been identified, including tension cracks along the seabed near the sides of the trench, “trench within a trench” configurations, and trench walls overhanging the riser.
The speed with which a SCR trench develops was assessed during the STRIDE JIP Phase III harbour tests where, over six weeks, the trench depth increased from 0.5 diameters to 1.2 diameters and the trench width increased to 2.5 diameters. If the harbour test riser had been allowed to continue to increase the size of the trench at the same rate over a year, the resulting trench would have been 6.6 diameters deep and 14 diameters wide. Comparing this with the ROV survey data where, after seven months, the Allegheny production riser trenches were 4.5 diameters wide and five diameters deep, it was concluded that the rate of trench propagation slows with time. This would suggest that, within a short time following installation, the riser will develop a trench of a reasonable size that will not significantly increase in size for the remainder of its lifetime.

The generic SCR trench profile can be implemented in SCR analysis using either spring or profiled surface elements. However, when the geometry of the SCR trench was compared with the TDP location study by Thethi and Moros (2001), it was observed that this profile could be further simplified depending on the vessel motions and the type of analysis conducted. For day-to-day vessel motions, that occur for 95% of the risers life, and vessel motions that are predominantly inline with the SCR, in the riser plane, the trench walls can be ignored and the trench can be adequately modelled using a sloping seabed. In contrast, for lateral extreme storm motions riser/trench interaction is likely to occur. However, since the bending is out-of-plane, the impact on SCR design was found to be negligible. It was concluded that the only case where riser/trench interaction is significant is during large lateral slow drift or second order motions, and even then only one side of the trench needs to be modelled, as shown in Figure 3.30, Section 3.4.4.

To assess the effect of the SCR trench on SCR maximum stress and fatigue loading, a FE model of a deepwater SCR in the Gulf of Mexico was constructed. The dynamic section of the SCR trench was modelled using a sloping rigid seabed with gradients between ±5%. The analysis results were compared to those generated using a rigid flat seabed and showed that, when the riser was either subject to static or extreme storm loads, the maximum stress at the TDP increased by less than 1.4%. This was considered insignificant and within modelling tolerances. When the SCR
was subject to fatigue loading the fatigue damage changed by up to 6%. However, when conducting fatigue analysis, a safety factor of ten is usually applied to the resulting fatigue damages, so the increase in fatigue damage due to seabed slope, associated with a trench, was also considered insignificant.

10.3 Pipe/Soil Suction

Reports from pipe laying contractors during the STRIDE JIP (2H Offshore, 2000) suggested that, when retrieving pipelines from clay seabeds, the force required to lift the pipe (which was in the form of a catenary) was higher than the support load during installation. This implies that a bond exists between a pipe resting on the seabed and the surrounding soil, which has been termed pipe/soil suction (Muga, 1968). Consequently, concern was raised regarding the effect of pipe/soil suction on SCRs and the reliability of analysis methods that, until now, have ignored it.

10.3.1 Full Scale Tests

To address pipe/soil suction, a series of full-scale tests were conducted, in order to demonstrate the influence of pipe/soil suction on riser bending moment. The tests showed that, when the riser experienced slow drift motions, the peak bending moment during a pull-out (with pipe/soil suction) could be twice that of a lay down (without pipe/soil suction). The tests also showed that the consolidation time (the rest time prior to a pull-out) could increase the pipe/soil suction force, and hence the increase in bending moment experienced by the riser. The number of previous pull-out tests experienced by the riser could reduce the pipe/soil suction force, and hence reduce the bending moment experienced by the riser.

Tests were also conducted to examine the increase in bending moments due to dynamic motions, such as those caused by day-to-day or extreme storm motions. It was observed that the change in peak bending moment in the riser due to dynamic motions was small.

Finite element analysis of the harbour test riser was conducted with ANSYS (Ansys Inc, 2000) with a pipe/soil interaction model developed by the writer, which
accounted for vertical soil stiffness and pipe/soil suction. The tri-linear pipe/soil suction curve was scaled from a series of 2D pipe/soil interaction tests conducted in STRIDE Phase II (2H Offshore, 1999b). The pull-out velocity, consolidation time and pipe weight were matched to those of the full scale harbour test riser experiments. From this analysis it was concluded that FE codes can be used to model pipe/soil suction accurately. However, the FE results are only as good as the pipe/soil model, and alternative pipe/soil suction curves would need to be developed for other risers.

10.3.2 Small Scale 2D Pipe/Soil Suction Tests

Small scale 2D pipe/soil suction tests were devised to improve the pipe/soil suction curve used in the FE analysis of the harbour test riser. These tests involved a pipe placed into a trench, then pulled out at different pull-out velocities, after different consolidation times and loads. The normalised shape of the pipe/soil suction curve was consistent for all tests and was similar to that published by Bostrom et al (1998).

The 2D pipe pull-out tests showed that increasing the pull-out velocity, consolidation time, or consolidation load, increases the maximum pipe/soil suction force and the break-out displacement. The effects of pull-out velocity on the maximum pipe/soil suction force was consistent with data published by Byrne & Finn (1978). It was also evident that increasing the number of prior cycles or having a loose trench compared to a close fitting trench decreased the maximum pipe/soil suction force and break out displacement.

In order to analyse the data from the STRIDE and CARISIMA JIP pipe/soil pull-out experiments, normalisation parameters were developed by the writer, based on work by Byrne and Finn (1978) and others, given in Section 5.3.3. The relationships observed between the normalised parameters form the basis of the pipe/soil suction model. The pipe/soil suction model takes into consideration the pull-out velocity, consolidation time and load, number of previous cycles, trench depth and cyclic loading, and uses empirical constants (derived by the writer and given in Chapter 6). These are used to calculate the limits of the pipe/soil suction curve; namely the maximum pipe/soil suction force and the break-out displacement. The empirical
10.3.3 Effect of Pipe/Soil Suction on SCRs

The pipe/soil suction model was used in an FE analysis of a deepwater SCR to confirm the observations from the harbour test riser experiments. The results showed that during a slow drift motion, where the riser was pulled up and away from the seabed, the maximum stress increased by 50%. However, during extreme storm and fatigue loading the maximum stress increased marginally by 0.7% and the fatigue damage changed by 0.9%. These results indicate that, for extreme storm and fatigue loading conditions, the effects of pipe/soil suction are negligible.

Pipe/soil suction may not have a large effect on SCRs during day-to-day operation, as the TDP motions are small and cyclic, and any pipe/soil suction force mobilised will dissipate. However there are instances during the life of a SCR where the effect of pipe/soil suction will be significant, and potentially damaging to the riser. Some examples are given below:

- **SCR or pipeline installation** – if a storm occurs during a SCR or pipeline installation, the pipe is disconnected from the vessel, and lowered to the seabed. After the storm has passed the SCR is retrieved from the seabed, and will mobilise pipe/soil suction, increasing the bending moment in the pipe and the tension load on the winch wire.

- **Slow drift vessel motions** – if the vessel supporting the SCR needs to be moved (e.g. to allow access to a subsea well by a second vessel) the SCR could be pulled out of the trench, mobilising pipe/soil suction and high vessel loads.

10.4 Vertical Downward Pipe/Soil Interaction

Examination of published literature showed a wide range of soil stiffness values that could be used in SCR analysis. Using experimental data from the STRIDE and CARISIMA JIPs the writer has determined a non-linear model that can be used in
pipe/soil interaction problems. However, current riser analysis FE codes limit the soil stiffness to a single value. Consequently, three different linear soil stiffnesses were identified as important in SCR analysis, namely: static, large displacement and cyclic soil stiffness.

Examination of the static soil stiffnesses for pipe penetration into a marine clay soil showed that many authors have attempted to re-write the bearing capacity equations for foundation embedment proposed by Terzaghi (1943), and developed increasingly complex formulations. However, the pipe penetration experimental data examined by the writer could be predicted using simple bearing capacity equations and the values of N given by Skempton (1951). Consequently, for static SCR analysis where penetration speed and consolidation time can be ignored, the writer has presented a method for calculating static soil stiffness based on bearing capacity equations. This has then been simplified to an equation that showed that soil stiffness is proportional to the undrained shear strength and external (or bearing) pipe diameter.

An assessment of pipe/soil interaction curves has been conducted which showed that the mobilisation distance (the distance over which the full soil resistance force was activated) is approximately 2.5% of the pipe diameter. This is smaller than the 10% given by Audibert et al (1984) for buried pipelines. The hyperbolic model (Kondner, 1963) was fitted to the pipe/soil unloading data from CARISIMA and using a mobilisation distance of 2.5% was shown to provide a good fit to the test data, while a mobilisation distance of 10% fitted the combined unloading and pipe/soil suction mobilisation curve. These values of mobilisation distance have then been used to develop a model for large displacement soil stiffness, which can be used to model the pipe/soil interaction in the analysis of an installed SCR.

To determine the cyclic soil stiffness for SCR analysis, the cyclic loading data recorded during the CARISIMA JIP has been examined. A method is presented in Chapter 7 for normalising cyclic pipe/soil interaction data into a parameter termed k\textsubscript{stiff}, which accounts for undrained shear strength, trench depth, pipe length and diameter, and provides a measure of normalised pipe/soil stiffness. Using Kondners (1963) hyperbolic model, an equation has been derived for the k\textsubscript{stiff} parameter, which
was shown to produce higher values of $k_{\text{stiff}}$ than 90% of those determined from the experimental data, Figure 7.17.

The $k_{\text{stiff}}$ parameter has been used to create a model for cyclic soil stiffness for pipe/soil interaction. This model calculates a value for $k_{\text{stiff}}$ using the mobilisation distance and the dynamic displacement of the pipe. Additional empirical parameters, such as the cyclic loading factor, have been developed to account for the number of previous cycles and the frequency of the oscillations (or interaction velocity). Using the hyperbolic model to calculate $k_{\text{stiff}}$ showed a range of cyclic soil stiffnesses between 200 to 2000 times the undrained shear strength, Equation (7.29).

A closed form seabed interaction model has been developed to calculate the effect of soil stiffness on the maximum riser stress and fatigue life. This showed that the value of soil stiffness used does not change the maximum stress at the TDP, but has a great effect on the fatigue damage. It also showed that high fatigue damage is generated in stiff soils while low fatigue damage is generated in soft soils. These results were then confirmed using finite element analysis, which also examined the effects of pipe/soil suction on maximum SCR stress and fatigue life.

The results indicate that most SCR analysis, except fatigue analysis, can be conducted using a rigid seabed, as soil stiffness does not have a great effect. Using a rigid seabed in fatigue analysis gives conservative results, as the high contact force at the TDP between the riser and seabed increases the peak stress, which reduces the number of cycles to failure, and lowers the fatigue life. Less conservative fatigue lives can be achieved using elastic seabeds with dynamic soil stiffness that spread the TDP reaction force along the TDZ and reduce the peak stress. However, care must be taken so that the soil stiffnesses used are not too low and unrepresentative.

10.5 Summary of Findings

A summary of the influences of the trench shape, soil stiffness and pipe/soil interaction, on SCR maximum stress and fatigue damage under different loading conditions, is given in Table 10.1. This shows that for most SCR analysis the effects of trench shape, soil stiffness and pipe/soil suction are small. However, pipe/soil
suction is significant during slow drift vessel motions or during pipe retrieval and a model for this has been created and presented in Chapter 6. Soil stiffness is important for fatigue analysis, and a dynamic soil stiffness model has been created and was outlined in Chapter 7. This model can be used in small and large displacement analysis and accounts for dynamic pipe displacement, hysteresis and the speed of the dynamic pipe/soil interaction. The cyclic model is based on the CARISIMA test data and is shown to be conservative.

<table>
<thead>
<tr>
<th>Design Criteria</th>
<th>Trench Shape</th>
<th>Soil Stiffness</th>
<th>Pipe/Soil Suction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Large Vessel Motions</td>
<td>Negligible</td>
<td>Negligible</td>
<td>Significant</td>
</tr>
<tr>
<td>Fatigue Damage</td>
<td>Negligible</td>
<td>Significant</td>
<td>Negligible</td>
</tr>
</tbody>
</table>

A summary of the original work covered in this thesis, which includes a number of pipe/soil interaction models formulated by the writer and that are now being used within the industry is given below:

- Generic trench model for SCR/trench interaction analysis (Chapter 3).
- Experimental data investigating the suction force between a pipe and a clay soil (Chapters 4 and 5).
- Pipe/soil suction model that accounts for pipe diameter, pull out speed, consolidation time and load, and soil plasticity index (Chapter 6). This provides a framework for implementing pipe/soil suction forces into finite element analysis codes.
- Non-linear static and dynamic vertical pipe/soil interaction models for pipe/seabed interaction (Chapter 7).
- Static and dynamic equations for linear soil stiffness (Chapter 7).
- Closed-form SCR TDZ model showing that fatigue damage increases with increasing soil stiffness (Chapter 8).
• FE analysis of a SCR that confirms the results of the above-mentioned closed-form SCR TDZ model (Chapter 9).

10.6 Recommendations for Further Work

During the course of this work a number of areas have been identified where further investigation to improve understanding may be warranted. These areas are summarised below:

• Trench Surveys – the trench surveys provided a range of different trenches for examination, although the soil data available was limited. It would be advisable to determine the impact of soil properties, such as plasticity index, on SCR trench shape and to improve understanding of trenching mechanisms.

• Pipe/Soil Suction – pull-out tests to examine pipe/soil suction have been conducted on two soft marine clays with plasticity indices, \( I_p \), of between 30% and 42%. High plasticity (\( I_p = 100\% \)) clay soils exist in offshore West Africa. This plasticity index was out of the range of the clays tested and the models developed may not be appropriate. Consequently, pull-out tests should be conducted on a high plasticity marine clay to verify (or otherwise) the applicability of the pipe/soil suction model for high plasticity clays.

• Soil Stiffness – the pipe/soil interaction model developed in Chapter 7 was non-linear, but because of limitations of existing riser FE codes, the soil stiffness models developed were linear. The effect of non-linear soil stiffness on SCRs should be assessed to determine the validity of the linear soil stiffness assumption.

• Soil Stiffness – The pipe/soil interaction models are considered to be conservative for SCR analysis because, inter alia, they use the undisturbed soil shear strength. It is recognised that the models do not cover all aspects of pipe/soil interaction and consequently there are areas where further test data and model development is required to reduce this conservatism. Suggested tests include cyclic tests to lower the minimum cyclic rate factor and examining a range of marine clays with different plasticity indices.
• SCR Studies – the FE study presented in this thesis consists of one SCR on a single soil. A wider range of studies using representative SCRs in different environments, water depths, and soil types is required to confirm the conclusions presented in this thesis.

• SCR Trench Study – small scale experiments of the lower portion of a SCR, similar to the Watchet Harbour test riser, to examine and confirm trenching mechanisms and rates observed in the ROV trench surveys.

• SCR Trench Development – examine trench development using finite element analysis with an elasto-plastic soil model to investigate how plastic soil behaviour influences the trench shape and stress distribution within the TDP.
11.0 PUBLICATIONS ASSOCIATED WITH THIS THESIS

During the research for this thesis the writer produced a number of documents related to pipe/soil interaction, steel catenary riser trenching mechanisms and riser design and analysis. Many of these documents were produced for phases 3 and 4 of the STRIDE JIP. A list of these references is given below.

The following documents were produced by the writer and published within the STRIDE JIP. They have been peer reviewed by the industry experts who participated in the STRIDE JIP:


The following is a list of the conference papers and journal articles produced by the writer. Copies of these papers are included in Appendix F.


The following is a list of papers authored or co-authored by the writer that cover topics including riser damping, vortex induced vibration and fatigue monitoring.


12.0 REFERENCES AND BIBLIOGRAPHY

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Winkler, E. (1867) – “Die Lehre von der Elastizität and Festigkeit” (The Theory of Elasticity and Stiffness), H. Dominicus, Prague, Czechoslovakia.


APPENDIX A  DERIVATION OF CATENARY EQUATIONS

A.1  Introduction

During feasibility studies the geometric properties and forces along an SCR need to be estimated quickly and accurately. This appendix presents the derivation of a series of equations that can be used to estimate the geometric properties and forces on an SCR.

The parameters used to describe a steel catenary riser are given below

- Cross-sectional riser pipe properties such as outer diameter, wall thickness, density of steel and density of the internal fluid, and hence the submerged mass per unit length of the riser.
- Height of the vessel/riser attachment point above sea level
- The hang off angle between the top of the riser and the horizontal axis

A.2  The Catenary Equations

The catenary zone of a steel catenary riser (SCR) extends from the riser/vessel connection to the touchdown point (TDP). Within the catenary zone the riser hangs freely under the action of its own uniformly distributed weight. This can be described by the equation by Leibniz (1691a) and (1691b) that is given below. This solution assumes that the riser is a cable structure with no bending stiffness and infinite axial stiffness and has no additional applied forces, such as current drag or buoyancy.

\[ H \frac{d^2 z}{dx^2} = m_s g \left[ 1 + \left( \frac{dz}{dx} \right)^2 \right]^{1/2} \]  (A.1)

where
- \( z \) is the vertical distance from the seabed to a point on the riser
- \( x \) is the horizontal distance between the TDP and a point on the riser
- \( m_s \) is the submerged mass per unit length
- \( g \) is the acceleration due to gravity
- \( H \) is the horizontal force in the riser, and the tension at the TDP
Equation (A.1) has a solution relating the vertical axis to the horizontal axis. This equation, which is called the catenary equation and is taken from Timoshenko (1965), is given below.

\[
z = \frac{H}{m_s g} \left[ \cosh \left( \frac{m_s g x}{H} \right) - 1 \right]
\]

(A.2)

A summary of the notation used in the equations above is given in Figure A.1.

By assessing the equilibrium of the catenary riser the top tension can be derived.

Timoshenko also showed that the length of the riser from the TDP, S, can be determined using the differential equation given below.

\[
\frac{dz}{dx} = \sinh \left( \frac{m_s g x}{H} \right)
\]

(A.4)

Substituting equation (A.4) into equation (A.3) and integrating with respect to x gives

\[
S = \frac{H}{m_s g} \sinh \left( \frac{m_s g x}{H} \right)
\]

(A.5)
Examining the equilibrium of forces on the riser it is observed that the tension at any point on the riser, $T$, can be found using the horizontal tension, $H$ and the mass of the riser from the TDP to the point in question, as shown in Figure A.1. This is expressed using the equation given below.

$$T = \sqrt{H^2 + (m_s g S)^2}$$  \hspace{1cm} (A.6)

Substituting equation (A.5) into the above equation and rearranging gives

$$T^2 = H^2 \left[ 1 + \sinh \left( \frac{m_s g x}{H} \right) \right] = H^2 \cosh \left( \frac{m_s g x}{H} \right)$$  \hspace{1cm} (A.7)

Substituting this into equation (A.2) and rearranging gives the following equation that shows that tension has a simple relationship with horizontal force.

$$T = H + m_s g z$$  \hspace{1cm} (A.8)

Another important property of an SCR is the slope of the riser, $\alpha$. Since SCRs are considered large non-linear structures small angle theory can not be used, and as such the gradient of the riser is equal to the tangent of the slope, as shown below.

$$\frac{dz}{dx} = \sinh \left( \frac{m_s g x}{H} \right) = \tan \alpha$$  \hspace{1cm} (A.9)

Rearranging this equation gives an equation for $x$ in terms of $\alpha$, $H$, $m_s$ and $g$.

$$x = \frac{H}{m_s g} \arcsinh (\tan \alpha)$$  \hspace{1cm} (A.10)

Substituting equation (A.9) into (A.5) produces the following relationship

$$\frac{H}{m_s g} = \frac{S}{\tan \alpha}$$  \hspace{1cm} (A.11)

This can then be used to simplify equation (A.2) and produces an equation for $z$ using the geometric terms of $x$, $S$ and $\alpha$.

$$z = \frac{S}{\tan \alpha} \left[ \cosh \left( \frac{x \tan \alpha}{S} \right) - 1 \right]$$  \hspace{1cm} (A.12)

Further substitutions can be conducted to show that both $z$ and $S$ can be defined by combinations of $x$ and $\alpha$ as shown below.

\hspace{1cm} A-3
Using the above equations a series of equations for SCR geometry can be derived relating the distance to the TDP from the vessel, \( x_{\text{TDP}} \), and the riser catenary length, \( S_R \), to the height of the riser attachment point above the seabed, \( z_A \), and the top angle, \( \alpha_{\text{TOP}} \). These equations are particularly useful in SCR analysis as design bases generally specify water depth and top angle, and are given below.

\[
x_{\text{TDP}} = z_A \frac{\arcsinh (\tan \alpha_{\text{TOP}})}{\cosh (\arcsinh (\tan \alpha_{\text{TOP}})) - 1}
\]

(A.15)

\[
S_R = z_A \frac{\tan \alpha_{\text{TOP}}}{\cosh (\arcsinh (\tan \alpha_{\text{TOP}})) - 1}
\]

(A.16)

where

\[\alpha_{\text{TOP}} = 90 - \theta\]

(A.17)

For completeness an equation relating \( S_R \) to \( x_{\text{TDP}} \) is given below.

\[
S_R = x_{\text{TDP}} \frac{\tan \alpha_{\text{TOP}}}{\arcsinh (\tan \alpha_{\text{TOP}})}
\]

(A.18)

\[\text{A.3 Bending Moment}\]

The bending moment at the TDP can be calculated from the curvature, \( k \), of the catenary riser, which for geometric non-linear systems (large deflections) is given below:

\[
k = \frac{d^2 z}{dx^2} \left[ 1 + \left( \frac{dz}{dx} \right)^2 \right]^{\frac{3}{2}}
\]

(A.19)

where

\[
\frac{d^2 z}{dx^2} = \frac{m_S g}{H} \cosh \left( \frac{m_S g x}{H} \right)
\]

(A.20)

Substituting equations (A.4) and (A.20) into (A.19) and rearranging gives
Using the standard relationship relating curvature to bending moment, shown below, an equation for the maximum bending moment can be derived

\[ k = \frac{M}{EI} \]  
(A.22)

\[ M = \frac{m_s g}{H} \left( \frac{1}{\cosh \left( \frac{m_s g x}{H} \right)} \right)^2 EI \]  
(A.23)

By differentiating the bending moment equation, a relationship for the shear force along the riser length can be derived.

\[ V = 2 \left( \frac{m_s g}{H} \right)^2 \frac{\sinh \left( \frac{m_s g x}{H} \right)}{\cosh \left( \frac{m_s g x}{H} \right)^3} EI \]  
(A.24)

A.4 Stresses at the TDP

At the TDP the displacement, \( x \), is equal to 0 and the above equations for curvature and bending moment simplify to the following.

\[ k_{TDP} = \frac{m_s g}{H} \]  
(A.25)

\[ M_{TDP} = -\frac{m_s g}{H} EI \]  
(A.26)

where

\[ k_{TDP} \] is the curvature at the TDP

\[ M_{TDP} \] is the moment at the TDP

The above equation for \( M_{TDP} \) can be changed into an equation relating the geometric SCR properties by substituting equation (A.11) into equation (A.26) as shown below.
Since it is assumed that the SCR is a cable structure the equation for shear force at the TDP reduced to 0.

**A.5 Equating Near and Far Offsets**

Initial riser sizing is based around the static nominal, near and far configurations. This requires that given the nominal configuration the near and far offsets and the respective configurations can be determined.

To determine the configuration of the SCR in either the near and far offset positions an equation is written summing the catenary length and the surface pipe length that is equal for the nominal and offset positions.

\[
S_R + S_N = S_{RF} + S_{NF}
\]  
(A.28)

Equating the horizontal distances gives:

\[
x_{TDP} + S_N + P_{Z_A} = x_{TDP-F} + S_{NF}
\]  
(A.29)
Taking equation (A.29) away from equation (A.28) gives:

$$S_R - x_{TDp} - Pz_A = S_{RF} - x_{TDp-F}$$  \hspace{1cm} (A.30)

Using equations (A.15) and (A.16), equation (A.30) can be written in terms of $z_A$. Note that for simplicity equations (A.15) and (A.16) will be written in the form

$$S_R = z_A \times f_1(\alpha)$$ and $$x_{TDp} = z_A \times f_2(\alpha)$$ respectively.

$$z_A f_1(\alpha) - z_A f_2(\alpha) - Pz_A = z_A f_1(\alpha_F) - z_A f_2(\alpha_F)$$  \hspace{1cm} (A.31)

Where

$$f_1(\alpha) = \frac{\tan \alpha}{\cosh(\text{arcsinh} [\tan \alpha]) - 1}$$  \hspace{1cm} (A.32)

$$f_2(\alpha) = \frac{\text{arcsinh}(\tan \alpha)}{\cosh(\text{arcsinh} [\tan \alpha]) - 1}$$  \hspace{1cm} (A.33)

Cancelling the $z_A$ terms gives:

$$f_1(\alpha) - f_2(\alpha) - P = f_1(\alpha_F) - f_2(\alpha_F)$$  \hspace{1cm} (A.34)

Since the unknown terms in equation (A.34) are both functions of $\alpha_F$, an approximation can be determined. Figure A.3 and equation (A.35) show the non-linear approximation which can be used solve equation (A.34) for $\alpha_F$. The correlation factor, $R^2$ is equal to 1, which indicates a very close approximation.

$$\alpha_F = 0.585 [f_1(\alpha_F) - f_2(\alpha_F)]^3 - 2.53 [f_1(\alpha_F) - f_2(\alpha_F)]^2 + 3.547 [f_1(\alpha_F) - f_2(\alpha_F)] - 0.03$$  \hspace{1cm} (A.35)

For calculations a convenient form of equations (A.29), (A.30) and (A.34) can be written:

$$\frac{S_R - x_{TDp}}{z_A} - P = f_1(\alpha_F) - f_2(\alpha_F) = \frac{S_{RF} - x_{TDp-F}}{z_A}$$  \hspace{1cm} (A.36)
A.6 Non-Dimensional Parameters

The ratio between $x_{\text{TDP}}$ and $z_A$ is plotted verses the top angle, $\theta$ in degrees between limits of $3^\circ$ and $20^\circ$, Figure A.4 and between $S_R$ and $z_A$ verses top $\theta$ in Figure A.5. Using linear regression a line of best fit can be plotted on top of the exact relationship and rules of thumb found for the ratios $x_{\text{TDP}}/z_A$ and $S_R/z_A$ with $\theta$, as shown below.

$$\frac{x_{\text{TDP}}}{z_A} = 0.04\theta + 0.1 \quad 3^\circ > \theta > 20^\circ \quad \text{(A.37)}$$

$$\frac{S_R}{z_A} = 0.022\theta + 0.98 \quad 3^\circ > \theta > 20^\circ \quad \text{(A.37)}$$
The Ratio Between $x_{TP}$ and $z_A$ From the Catenary Equations

\[ y = 0.0405x + 0.0994 \]
\[ R^2 = 0.999 \]

Figure A.4 – Top Angle of SCR with $x_{TP}/z_A$ Ratio

The Ratio Between $S_R$ and $z_A$ From the Catenary Equations

\[ y = 0.0219x + 0.9776 \]
\[ R^2 = 0.9977 \]

Figure A.5 – Top Angle of SCR with $S_R/z_A$ Ratio
A.7 Comparison Between the Catenary Equations and FEA

A comparison between the catenary equations and FEA was conducted to show that the static results from both methods are similar. A summary of the input parameters is given in Table A.1.

The results from the comparisons are given in Figure A.6 to Figure A.10 and are summarised in Table A.2. The static shape of the SCR is given in each plot. The effective tension, bending moment and von Mises stress to yield stress ratio are given in Figure A.7, Figure A.9 and Figure A.10 respectively show a good correlation between the catenary equations and the FEA. The shear force shown in Figure A.8 does not show good correlation. This is because the catenary equations assume that the SCR has zero stiffness and only a tension component at the TDP. The shear force has been interpolated from the bending moment data.

The von Mises stress to yield stress ratio plot shown in Figure A.10 shows two peaks in von Mises stress, one at the vessel/SCR connection point, the other at the TDP. From the tension and bending moment plots in Figures A.7 and A.9 respectively, the vessel/SCR connection is shown to be tension dominated while the TDP is shown to be bending dominated.
### Table A.1 – SCR Parameters Used for Static Comparison

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Gulf of Mexico SCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outer Diameter, ( D_o )</td>
<td>0.324m (12(1/4))”</td>
</tr>
<tr>
<td>Wall Thickness, ( t )</td>
<td>0.0205m (0.8”)</td>
</tr>
<tr>
<td>Inner Diameter, ( D_i )</td>
<td>0.283m (11.1”)</td>
</tr>
<tr>
<td>Second Moment of Area, ( I )</td>
<td>(2.26 \times 10^4 m^4)</td>
</tr>
<tr>
<td>Coating Thickness, ( t_E )</td>
<td>50mm (2.0”)</td>
</tr>
<tr>
<td>Height of the Attachment Point, ( z_A )</td>
<td>1600m (5249ft)</td>
</tr>
<tr>
<td>Top Angle to Horizontal, ( \theta )</td>
<td>12.0°</td>
</tr>
<tr>
<td>Steel Density, ( \rho_S )</td>
<td>7850 kg/m(^3) (8411lb/ft(^3))</td>
</tr>
<tr>
<td>External Coating Density, ( \rho_E )</td>
<td>700 kg/m(^3) (75lb/ft(^3))</td>
</tr>
<tr>
<td>Internal Fluid Density, ( \rho_F )</td>
<td>800 kg/m(^3) (86lb/ft(^3))</td>
</tr>
<tr>
<td>In Service Weight in Water, ( m_s )</td>
<td>100 kg/m (67lb/ft)</td>
</tr>
<tr>
<td>Bending Stiffness, ( E I )</td>
<td>(4.67 \times 10^7 \text{Nm}^2)</td>
</tr>
<tr>
<td>Seabed</td>
<td>Rigid surface</td>
</tr>
</tbody>
</table>

### Table A.2 – Comparison between Hand Calculations and Finite Element Analysis

<table>
<thead>
<tr>
<th>Variable</th>
<th>Catenary Equations</th>
<th>Finite Element Analysis</th>
<th>Difference</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height of Attachment point above the seabed</td>
<td>1600.0m</td>
<td>1600.0m</td>
<td>0.0m</td>
<td>1.00</td>
</tr>
<tr>
<td>Top angle</td>
<td>12.0°</td>
<td>12.0°</td>
<td>0.0°</td>
<td>1.00</td>
</tr>
<tr>
<td>Distance to TDP from vessel</td>
<td>946.1m</td>
<td>952.1m</td>
<td>6.0m</td>
<td>1.006</td>
</tr>
<tr>
<td>Riser length</td>
<td>1975.8m</td>
<td>1982.7m</td>
<td>6.9m</td>
<td>1.003</td>
</tr>
<tr>
<td>Top tension</td>
<td>1984kN</td>
<td>1968kN</td>
<td>16kN</td>
<td>0.992</td>
</tr>
<tr>
<td>Bottom tension</td>
<td>412.5kN</td>
<td>410.5kN</td>
<td>2kN</td>
<td>0.995</td>
</tr>
<tr>
<td>Bending moment at TDP</td>
<td>-111.3kNm</td>
<td>-109.6kNm</td>
<td>1.7kNm</td>
<td>0.985</td>
</tr>
</tbody>
</table>
Figure A.6 – Example of Slope along an SCR

Figure A.7 – Example of Effective Tension along an SCR
Example of Shear Force Along a Steel Catenary Riser
12.75in OD, 1800m Water Depth, Spar Vessel, 12° Top Angle
Internal Fluid Oil

Figure A.8 – Example of Shear Force along an SCR

Example of Bending Moment Along a Steel Catenary Riser
12.75in OD, 1800m Water Depth, Spar Vessel, 12° Top Angle
Internal Fluid Oil

Figure A.9 – Example of Bending Moment along an SCR
Figure A.10 – Example of von Mises Stress to Yield Ratio along an SCR

A.8 References


APPENDIX B SUPPLEMENTARY INFORMATION

B.1 Introduction

This appendix contains supplementary information to the literary review that was either derived by the writer or additional to the information given in Chapter 2.

B.2 Bearing Width

Bearing load is generally calculated as bearing load per unit length (kN/m/m) and L is taken as 1.0m. For a pipe in a shallow trench \((z < \frac{1}{2} D)\) the bearing width is less than the diameter of the pipe as shown in Figure B.1. An equation for \(B\) can be developed using Pythagoras’ theorem and is given below.

\[
\frac{1}{2} B = \sqrt{r^2 - (r-z)^2}
\]  

(B.1)

![Figure B.1 - Bearing Width of a Pipe in a Trench less than \(\frac{1}{2} D\)](image)

Equation (B.1) can be simplified and rearranged in terms of \(z\) and \(D\) as shown below.

\[
B = 2\sqrt{Dz - z^2}
\]  

(B.2)

B.3 Settlement and Consolidation

After initial static penetration of the pipe, the pipe will continue to sink into the soil due to consolidation. The long term penetration of the pipe, assuming there is no
dynamic loading, can be determined from the summation of the initial static penetration and the time dependant settlement theory, Craig (1996), as below.

\[ z_T = z_i + m_v \Delta \sigma' B U \]  

(B.3)

where

- \( z_T \) is the total settlement
- \( z_i \) is the immediate static penetration
- \( m_v \) is the coefficient of volume compressibility
- \( \Delta \sigma' \) is the change in stress
- \( U \) is the average degree of consolidation that varies between 0 and 1

The average degree of consolidation is dependant on the time between the application of the initial load and the time at which the penetration is required. The equations that describe how \( U \) changes with time are given below.

\[ T_r = \frac{\pi}{4} U^2 \quad U < 0.6 \]  

(B.4)

\[ T_r = -0.933 \log(1 - U) - 0.085 \quad U > 0.6 \]  

(B.5)

where

- \( T_r \) is the time factor defined as,

\[ T_r = \frac{c_v t}{d^2} \]  

(B.6)

where

- \( c_v \) is the coefficient of consolidation
- \( t \) is the time at which the pipe penetration is required
- \( d \) is the drainage distance, equivalent to the pipe diameter

**B.4 Degradation Factor**

The degradation of the soil resistance force due to cycling was examined by Idriss et al (1978). They related the reduction in the Young’s modulus of the soil to the number of cycles and the amplitude of the developed shear strain. This relative reduction was expressed as a degradation factor, \( D_E \), as shown below.

\[ D_E = \frac{E_n}{E_i} \]  

(B.7)
where

\[ E_n \] is the Young's modulus of the soil after n cycles

\[ E_i \] is the Young's modulus of the soil for the first cycle

### B.5 References


APPENDIX C  TEST DATA FROM HARBOUR TEST RISER

C.1  Introduction

The appendix presents a summary of the tension and bending moment test data recorded during the STRIDE phase 3 harbour test riser experiments. Details of the tests are presented within the thesis, while further information can be found in the STRIDE JIP reports published by 2H Offshore Engineering Ltd. It is not the intention of the writer to present a complete summary of the harbour test riser data, rather present additional data to that given within the thesis to help support the observations made.

This appendix has the following structure:

- Summary of selected Tests
- Vertical pull out experiments
- Vertical cyclic experiments
- Analysis results of Harbour Test Riser

C.2  Summary of Selected Tests

Bending moment and tension timetraces for the pull up, lay down and dynamic tests used within this thesis are given below. Each test is presented on a separate page, and each page is divided into two columns. The left hand column shows timetraces of actuator position, in-plane bending moment, out-of-plane bending moment and riser tension. The right hand column shows plots of in-plane bending moment, out-of-plane bending moment and riser tension with actuator position. A summary of the tests presented in given in Table C.1.
<table>
<thead>
<tr>
<th>Test Number</th>
<th>Test Corridor</th>
<th>Test Type and Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-5</td>
<td>Natural Trench</td>
<td>Dynamic @</td>
</tr>
<tr>
<td>2-6</td>
<td>Dynamic</td>
<td></td>
</tr>
<tr>
<td>2-9</td>
<td>Dynamic</td>
<td></td>
</tr>
<tr>
<td>2-10</td>
<td>Pull out test</td>
<td></td>
</tr>
<tr>
<td>2-11</td>
<td>Lay down test</td>
<td></td>
</tr>
<tr>
<td>2-12</td>
<td>Dynamic</td>
<td></td>
</tr>
<tr>
<td>3-5</td>
<td>Artificial Deepened Trench</td>
<td>1&lt;sup&gt;st&lt;/sup&gt; Pull out test in series</td>
</tr>
<tr>
<td>3-5A</td>
<td></td>
<td>2&lt;sup&gt;nd&lt;/sup&gt; Pull out test in series</td>
</tr>
<tr>
<td>3-5B</td>
<td></td>
<td>3&lt;sup&gt;rd&lt;/sup&gt; Pull out test in series</td>
</tr>
<tr>
<td>3-5C</td>
<td></td>
<td>4&lt;sup&gt;th&lt;/sup&gt; Pull out test in series</td>
</tr>
<tr>
<td>3-5D</td>
<td></td>
<td>5&lt;sup&gt;th&lt;/sup&gt; Pull out test in series</td>
</tr>
<tr>
<td>3-5E</td>
<td></td>
<td>6&lt;sup&gt;th&lt;/sup&gt; Pull out test in series</td>
</tr>
<tr>
<td>3-6</td>
<td></td>
<td>Lay down test</td>
</tr>
<tr>
<td>4-1</td>
<td>Backfilled Trench</td>
<td>Pull out test</td>
</tr>
<tr>
<td>4-2</td>
<td></td>
<td>Lay down test</td>
</tr>
<tr>
<td>5-6</td>
<td></td>
<td>Pull out test</td>
</tr>
<tr>
<td>5-7</td>
<td></td>
<td>Lay down test</td>
</tr>
<tr>
<td>5-12</td>
<td></td>
<td>Dynamic</td>
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<td>5-13</td>
<td></td>
<td>Dynamic</td>
</tr>
<tr>
<td>5-14</td>
<td></td>
<td>Dynamic</td>
</tr>
</tbody>
</table>
Figure C.1 – Summary of Test Data from Test 2-5
Figure C.2 – Summary of Test Data from Test 2-6
Figure C.3 – Summary of Test Data from Test 2-9
Figure C.4 – Summary of Test Data from Test 2-10
Test 2-11
Lay Down Test in Test Corridor 2
(Natural Trench)

Figure C.5 – Summary of Test Data from Test 2-11
Figure C.6 – Summary of Test Data from Test 2-12
Figure C.7 – Summary of Test Data from Test 3-5
Test 3-5A
Pull Up Test in Test Corridor 3
(Artificially Deepened Trench)

Figure C.8 – Summary of Test Data from Test 3-5A
Figure C.9 – Summary of Test Data from Test 3-5B
Figure C.10 – Summary of Test Data from Test 3-5C
Figure C.11 – Summary of Test Data from Test 3-5D
Test 3-5E
Pull Up Test in Test Corridor 3
(Artificially Deepened Trench)

Figure C.12 – Summary of Test Data from Test 3-5E
Figure C.13 – Summary of Test Data from Test 3-6
Test 4-1
Pull Up Test in Test Corridor 4
(Backfilled Trench)

Figure C.14 — Summary of Test Data from Test 4-1
Figure C.15 – Summary of Test Data from Test 4-2
Figure C.16 – Summary of Test Data from Test 5-6
Figure C.17 – Summary of Test Data from Test 5-7
Figure C.18 – Summary of Test Data from Test 5-12
<table>
<thead>
<tr>
<th>Test 5-13</th>
<th>Dynamic Test in Test Corridor 5 (Rigid Seabed)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Actuator Position Vs Time</strong></td>
<td><img src="image1" alt="Graph" /></td>
</tr>
<tr>
<td><strong>In-Plane Bending Moment Vs Time</strong></td>
<td><img src="image2" alt="Graph" /></td>
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<tr>
<td><strong>Out-of-Plane Bending Moment Vs Time</strong></td>
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</tr>
<tr>
<td><strong>Tension Vs Time</strong></td>
<td><img src="image4" alt="Graph" /></td>
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</tbody>
</table>

---

**Figure C.19 – Summary of Test Data from Test 5-13**

---
Figure C.20 – Summary of Test Data from Test 5-14
C.3  Vertical Pull Out

Typical tension and bending moment traces with actuator location for strain gauges A, C, D, F, J, K and M on test corridors 2 (naturally formed trench), 3 (artificially deepened trench), 4 (back filled trench), and 5 (rigid seabed) are shown in Figures C.21, C.22, C.23 and C.24 respectively. A comparison of the change in bending moment with actuator position for strain gauge locations D for all test corridors is given in Figure C.25.
Figure C.21 – Comparison of Pull Up and Lay Down Bending Moments
Natural Trench (Test Corridor 2)
Figure C.22 – Comparison of Pull Up and Lay Down Bending Moments
Hand Dug Trench (Trench Corridor 3)
Figure C.23 – Comparison of Pull Up and Lay Down Bending Moments Backfilled Trench (Test Corridor 4)
Figure C.24 – Comparison of Pull Up and Lay Down Bending Moments Rigid Seabed (Test Corridor 5)
Comparison of Bending Moments At Strain Gauge D for All Test Corridors

Figure C.25 – Comparison of Pull Up and Lay Down Bending Moments on All Test Corridors at Strain Gauge Location D
C.4 Vertical Cyclic Motions

Test data from tests with vertical cyclic motions for tests conducted in the open trench and rigid seabed are presented in the sections below.

C.4.1 Open Trench

The top tensions and bending moment trances with actuator position when the riser was in the open trench are given in the following figures. A summary of the figures presented is given in Table C.2.

Table C.2 – Summary of Presented Cyclic Tests

<table>
<thead>
<tr>
<th>Actuator Location</th>
<th>In Air or Water</th>
<th>Test Number</th>
<th>Tension</th>
<th>Bending Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal</td>
<td>Air</td>
<td>2-12</td>
<td>Figure C.26</td>
<td>Figure C.27</td>
</tr>
<tr>
<td>Near</td>
<td>Air</td>
<td>2-9</td>
<td>Figure C.28</td>
<td>Figure C.29</td>
</tr>
<tr>
<td>Nominal</td>
<td>Water</td>
<td>2-5</td>
<td>Figure C.30</td>
<td>Figure C.31</td>
</tr>
<tr>
<td>Near</td>
<td>Water</td>
<td>2-6</td>
<td>Figure C.32</td>
<td>Figure C.33</td>
</tr>
</tbody>
</table>
Figure C.26 – Tension of Dynamic Motions at Near in Open Trench in Air

Figure C.27 – Bending Moment of Dynamic Motions at Near in Open Trench in Air
Figure C.28 – Tension of Dynamic Motions at Nominal in Open Trench in Air

Figure C.29 – Bending Moment of Dynamic Motions at Nominal in Open Trench in Air
Tension with Actuator Motion,
Test 2-5 Dynamic Motions at Near, Open Trench in Water

Figure C.30 – Tension of Dynamic Motions at Near in Open Trench in Water

Bending Moment with Actuator Motion,
Test 2-5 Dynamic Motions at Near, Open Trench in Water

Figure C.31 – Bending Moment of Dynamic Motions at Near in Open Trench in Water
Figure C.32 – Tension of Dynamic Motions at Nominal in Open Trench in Water

Figure C.33 – Bending Moment of Dynamic Motions at Nominal in Open Trench in Water
C.4.2 Rigid Seabed

The top and bottom tensions and bending moments recorded during the cyclic test numbers 5-12, 5-13 and 5-14, which were conducted on the rigid seabed at Near, Nominal and Far actuator location respectively are presented in the figures below. Time traces of the top tensions are given in Figure C.34, and the bottom tensions in Figure C.35. Traces of the top tensions with actuator location are given in Figure C.36. Timetraces of bending moments at actuator locations at near, nominal and far are given in Figure C.37, Figure C.38 and Figure C.39 respectively. Traces of bending moment with actuator location at near nominal and far actuator locations for strain gauges A and C are given in Figure C.40 and strain gauges D and M in Figure C.41.

Figure C.34 – Dynamic Top Tensions on Rigid Seabed in Air
Figure C.35 – Dynamic Bottom Tensions on Rigid Seabed in Air

Figure C.36 – Dynamic Tensions with Actuator Amplitude
Figure C.37 – Timetrace of Bending with Dynamic Near Motions

Figure C.38 – Timetrace of Bending with Dynamic Nominal Motions

Figure C.39 – Timetrace of Bending with Dynamic Far Motions
Summary of In Plane Bending Moment  
Dynamic Motions at Near, Nominal and Far on Rigid Seabed in Air  
Strain Gauge Locations A and C

Figure C.40 – Bending Moment with Actuator Motion

Summary of In Plane Bending Moment  
Dynamic Motions at Near, Nominal and Far on Rigid Seabed in Air  
Strain Gauge Locations D and M

Figure C.41 – Bending Moment with Actuator Motions
C.5 Analysis Results of Harbour Test Riser

Comparisons between the analysis results and the experimental data from the harbour test riser are presented in this section. Traces of top tension and bending moment with actuator position for the open trench are given in Figures C.42 and C.43, for the back filled trench in Figures C.44 and C.45 and for the rigid seabed in Figures C.46 and C.47. Analytical bending moment envelopes with bending moment ranges from the experimental data are presented for the open trench, the artificially deepened trench, the back filled trench and the rigid seabed in Figure C.48, Figure C.49, Figure C.50 and Figure C.51 respectively.
Figure C.42 – Comparison of Analytical Top Tensions with Open Trench Test Data

Figure C.43 – Comparison of Analytical Bending Moments with Open Trench Test Data
Figure C.44 – Comparison of Analytical Top Tensions with back Filled Trench Test Data

Figure C.45 – Comparison of Analytical Bending Moments with Back Filled Trench Test Data
Figure C.46 – Comparison of Analytical Top Tensions with Rigid Seabed Test Data

Figure C.47 – Comparison of Analytical Bending Moments with Rigid Seabed Test Data
Figure C.48 – Comparison of Analytical Bending Moment Envelope with Open Trench Test Data

Figure C.49 – Comparison of Analytical Bending Moment Envelope with Artificially Deepened Trench Test Data
Figure C.50 – Comparison of Analytical Bending Moment Envelope with Back Filled Trench Test Data

Figure C.51 – Comparison of Analytical Bending Moment Envelope with Rigid Seabed Test Data
APPENDIX D  CARISIMA TEST DATA

D.1 Introduction

Marintek conducted the CARISIMA JIP at the same time that 2H were conducting STRIDE JIP phases III and IV. The objectives of the CARISIMA JIP was to examine the effect of pull-out resistance on a pipe in a clay soil and create a model which could be programmed into the Marintek riser finite element analysis code. In addition to the pull-out tests they conducted cyclic pipe/soil interaction and pipe re-penetration experiment, however these tests were conducted mainly to see if they effected the soil suction forces. The data presented within this appendix is a summary of the CARISIMA tests conducted with some preliminary analysis that was conducted by the author.

D.2 Description of CARISIMA Test Rig

D.2.1 CARISIMA Test Rig Properties

The CARISIMA 2D test rig used in the experiments consists of a pipe suspended over a test tank containing an artificially consolidated clay, Figure D.1. Two pipes were used within the tests, a large pipe 0.219m in diameter and a small pipe 0.1016m in diameter. The pipes were rigid steel sections rigidly connected to the bottom of the actuator by an H beam, Figure D.2. The pipe (H beam) was actuated using two servo-hydraulic pistons on a feed back loop controlled by a PC. These allow the pipe to be lowered into the clay and subsequently pulled out in both the vertical and lateral directions. The pipe force, displacement and acceleration in both the lateral and vertical axis were measured and stored on an additional PC. A schematic of the CARISIMA test rig is given in Figure D.3 and a summary of the pipe parameters given in Table D.1.
Figure D.1 – Photograph of the CARISIMA Test Rig – End View

Figure D.2 – Photograph of the CARISIMA Test Pipe
D.2.2 Instrumentation

The CARISIMA actuator was able to log force, displacement and acceleration in both the vertical and horizontal axis. The force was measured using a Interface 1210AF-5KN-B load cell which had a calibrated range of ±5kN and a linear error of 0.04%. The stroke was measured using a MTS Temposonic III displacement sensor with a calibrated range of ±500mm and a linear error of 0.02%. The acceleration were measured with a Sundstrand data QA-1400 accelerometer with a calibrated range of ±0.75G and a linear error of 20μG/G^2. On the base of the test pipes were a set of 3 XPM5-2-G-HA earth pressure sensors with a calibrated range of -20 to +150 kPaG and a linear error of 0.35%.

Figure D.3 – Schematic of CARISIMA Test Rig, Marintek (2000b)
The data from the measuring instrumentation was collected using a 8 channel programmable amplifier (DIFA PDA) and a 8 channel filter (DIFA PDF) connected to a Fluke NetDAQ 2645A data logger which was programmed and controlled by a Compaq desktop computer. This enabled all of the instruments to be logger simultaneously at a frequency of 100Hz and filtered to 40Hz.

D.2.3 Data Processing
Marintek processed the raw data from the tests using Matlab program written specifically for these tests, Marintek (2000)b. The raw data was first filtered using a low pass filter to remove any noise. Then the force was corrected for inertia effects using the data from the accelerometers. The final step removed any phase lag caused by the analogue filtering process.

D.2.4 Soil Preparation
The CARISIMA tests were conducted on both the Onsoy and Watchet Harbour clays. The clays were prepared for the tests by thoroughly mixing each sediment with seawater, and then pouring them into 4 separate 5m x 2m steel tanks to a thickness of 360mm (3 for the Onsoy clay, 1 for the Watchet Harbour clay). The test bins were lined with a heavy tarpaulin, on top of which was placed a filter layer to allow free drainage. The filter material had a plastic core that prevented the filter being crushed under the weight of the soil. A similar filter layer was then placed on top of the clay, upon which water proof plywood was placed. This acted to spread the subsequent consolidation loads evenly across the soil and allow free drainage from the top surface of the clay. The filter layer was removed prior to the testing.

Consolidation of the Onsoy clay was conducted in 5 stages. The first stage was to lay a steel plate over the waterproof plywood layer. This was left for approximately 3 days after which 6 empty 1000litre water tanks were placed on top of the steel plate. This was then left for approximately 4 days. During the 3rd consolidation step the water tanks were evenly filled with water so that the total applied vertical stress was exactly 5kPa. The clay was then left for approximately 10days. The water tanks were then evenly filled so the applied vertical stress was 10kPa and then left to consolidation for 35days. After this the water tanks, steel, plywood and topmost filter
layers were removed and the clay was left to swell before the tests were conducted. The resulting ‘seabed’ was submerged beneath approximately 120mm of seawater. The consolidation procedure of the Watchet Harbour clay was similar to that of the Onsoy clays except that the clay was left to consolidate under load for approximately twice as long. This took approximately 3 months.

D.2.5 Geotechnical Parameters

The geotechnical parameters of the artificially consolidated Onsoy and Watchet Harbour clays used in the CARISIMA testing were measured by NGI and are given in Table A2.1. Individual shear strength parameters for each test location are given with the test data in the following sections. The geotechnical parameters show that the Watchet Harbour clay has a higher plasticity index, 42% compared to 30%, and has almost twice the undrained shear strength. In addition the coefficient of consolidation is twice as high for the Watchet Harbour clay indicating that the soil required longer to reach the same level of consolidation as the Onsoy clay.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Onsoy Clay</th>
<th>Watchet Harbour Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Test Beds</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Plasticity Index, Ip</td>
<td>30 %</td>
<td>42 %</td>
</tr>
<tr>
<td>Water Content</td>
<td>60-65%</td>
<td></td>
</tr>
<tr>
<td>Sensitivity of Clay</td>
<td>3.4</td>
<td>2.5</td>
</tr>
<tr>
<td>Undrained Shear Strength at Surface</td>
<td>1.5 kPa</td>
<td>3 kPa</td>
</tr>
<tr>
<td>Undrained Shear Strength Gradient</td>
<td>12.5 kPa/m</td>
<td>15 kPa/m</td>
</tr>
<tr>
<td>Undrained Shear Strength at 0.1m Depth (≈1D)</td>
<td>≈3.0 kPa</td>
<td>≈4.5 kPa</td>
</tr>
<tr>
<td>Coefficient of Consolidation</td>
<td>0.5 m²/year</td>
<td>0.2 m²/year</td>
</tr>
</tbody>
</table>

Table D.1 – Comparison of Soil Parameters from CARISIMA reports
In addition to these pipe/soil interaction tests CARISIMA was also evaluating a T-Bar apparatus, NGI (2000), for soil shear strength measurement. This instrument consists of a small horizontal bar (20mm diameter, 125mm long) connected to a vertical shaft (12mm diameter, about 500mm long) that was connected to a load cell. The apparatus was then pushed into the clay and the force and displacement measured. From this data the undrained shear strength of the clay could be determined. These were compared to the shear vane and triaxial tests and the correlation was found to be good. All of the shear strengths reported by Marintek were found using the T-Bar apparatus.

D.3 Overview of Tests

D.3.1 Overview of Tests Conducted

Four types of tests were conducted during CARISIMA, an outline of which is given below:

- Penetration – these were the initial test conducted to push the selected pipe into the clay. The pipe was pushed vertically into the soil at speeds of 0.03mm/s (small pipe) and 0.065mm/s (large pipe). These tests can be used to evaluate the bearing capacity $N_c$ factors and the backbone curve.

- Vertical Pull Out – these were the main focus of the testing campaign and are used to evaluate the effect of soil suction. After the pipe has been left to consolidate for a prescribed length of time under a known load of 100N (small pipe) or 625N (large pipe) the pipe is pulled vertically out of the trench at different speeds. These tests will be used to show the effect of pull out speed, consolidation time and cyclic loading on the mobilised soil suction force.

- Lateral Pull Out – these tests were conducted to examine the force/displacement curve as the pipe was pulled through (the pipe was fixed vertically) or over (the pipe was allowed to move vertically) the trench wall.

- Cyclic – these tests were conducted during the vertical pull out tests, prior to either consolidation or pull out. They will be used to show the hysteresis effect on the backbone curve and develop a soil stiffness model.
A total of 34 soil suction (14 using the Onsoy clay and 20 using the Watchet Harbour Clay) and 20 lateral (17 using the Onsoy clay and 3 using the Watchet Harbour clay) pull out tests were conducted.

The CARISIMA test parameters, such as push in speeds, pull out velocities and consolidation times were all scaled by the pipe diameters. This was done so that the forces and displacements generated during the tests from both large and small pipes would be directly comparable.

D.3.2 Test Procedure
All of the tests conducted typically followed the same test procedure. The pipe was pushed into the clay using a very low speed moving to a penetration depth of half a diameter over 1 hour. On most tests the pipe was then cycled for a period of time. On all tests the pipe was left under a constant load of 100N for the small pipe or 625N for the large pipe for a prescribed consolidation time. The actuator would then conduct the test, pulling the pipe vertically away from the soil or laterally into the trench wall. The pipe was then scraped clean and prepared for the next test.

Each of the pipe/soil interaction tests was conducted on virgin areas of clay. Remoulded clays were created from the virgin soil by mixing the section of the test bed directly underneath the pipe after the penetration procedure was completed.

The CARISIMA tests were conducted in two phases on two different clays. Phase I of the CARISIMA tests used both of the test pipes for both soil suction and lateral pull out tests on the 3 Onsoy clay test beds. The CARISIMA II tests used only the small test pipe since there was only 1 Watchet Harbour clay test bed, and in the earlier CARISIMA I tests, both test pipes had shown consistent results (see results below).
D.3.3 Normalisation Analysis

The normalisation parameters used to analyse the CARISIMA test data are the same as those developed for the STRIDE work and are defined in Section 5.0. A summary of the normalisation parameters is given below.

\[
\frac{V}{D} \cdot \frac{Q_s}{LDS_y} \cdot \frac{\Delta_B}{D} \cdot \frac{F_c}{LD} \times \sqrt{\frac{c_v t}{D^2}}
\]

where

\( V \) pull out or push in velocity
\( D \) outer pipe diameter
\( Q_s \) vertical force
\( L \) length of the pipe
\( S_u \) undrained shear strength of the clay soil
\( \Delta_B \) break out displacement
\( F_c \) pipe weight and pre load
\( c_v \) coefficient of consolidation
\( t \) length of time of consolidation

D.4 Vertical Penetration Tests

This section details the penetration tests conducted during both phases of the CARISIMA testing.

Penetration tests were conducted using both the small and large diameter pipes. CARISIMA I (Onsoy Clay) tested using a combination of pull out velocities on either virgin or remoulded clays. An overview of these is given in Table D.2. CARISIMA II (Watchet Harbour Clay) only used the small diameter pipe and only penetrated using 0.03mm/s. However the pipe was pushed twice into each trench, first to 0.5D and then after the pull out test to either 1.0D or 1.5D. In addition a cyclic loading test was conducted prior to penetration that shows the effect of hysteresis on the backbone curve. These are summarised in Table D.3.
Table D.2 – CARISIMA I (Onsoy Clay) Penetration Test Matrix

<table>
<thead>
<tr>
<th>Pipe Size</th>
<th>Push In Velocity</th>
<th>0.03mm/s</th>
<th>0.065mm/s</th>
<th>10mm/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Small Pipe 0.0106m Do</td>
<td>1 – 6, 8, 11, 13, 14</td>
<td>-</td>
<td>-</td>
<td>7</td>
</tr>
<tr>
<td>Large Pipe 0.2191m Do</td>
<td>-</td>
<td>9, 10, 12</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Notes: Tests in Red are conducted on Remoulded clay

Table D.3 – CARISIMA II (Watchet Harbour Clay) Penetration Test Matrix

<table>
<thead>
<tr>
<th>Penetration Depth and Starting Location</th>
<th>Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>0D - 0.25D</td>
<td>6-1</td>
</tr>
<tr>
<td>0D - 0.5D</td>
<td>1-1, 2-1, 3-1, 4-1, 5-1, 7-1, 8-1, 9-1, 10-1</td>
</tr>
<tr>
<td>0.25D - 1.5D</td>
<td>6-2</td>
</tr>
<tr>
<td>0.5D - 1.0D</td>
<td>1-2, 2-2, 3-2, 10-2</td>
</tr>
<tr>
<td>0.5D - 1.5D</td>
<td>4-2, 5-2, 7-2, 8-2, 9-2</td>
</tr>
</tbody>
</table>

Notes: All test conducted with a penetration speed of 0.03mm/s Test in Bold are conducted after cycling

Penetration tests were conducted using both the small (0.1016m) and large (0.2191m) diameter pipes. The penetration speeds used in the CARISIMA I (Onsoy Clay) were 0.03mm/s (V/D = 0.0003) and 0.065mm/s (V/D = 0.0003) for the small and large diameter pipes respectively. In addition one CARISIMA I penetration test using the small diameter pipe was conducted with a fast penetration speed of 10mm/s (V/D = 0.098) between 0.1z/D and 0.5z/D. CARISIMA II (Watchet Harbour Clay) used the small diameter pipe with a penetration speed of 0.03mm/s.

Initial comparisons are conducted using the small diameter pipe tests from CARISIMA phase I. The normalised force-displacement curves of these penetration tests all follow the same trend as shown in Figure A.4. The normalised force at zero
is zero as the pipe is just touching the seabed. As the pipe is penetrated into the soil the normalised force increases to approximately half of the final normalised force over a displacement of 0.1z/D. The force then increases almost linearly to the maximum normalised force of 4.5 as the displacement increases to 0.5z/D.

Further comparisons are made between the normalised force\displacement curves from the small and large diameter pipes with the same normalised penetration speed. CARISIMA phase 1 tests 02 and 10 are compared, as these are representative of the small and large diameter penetration tests respectively. The comparison is given in Figure A.5 and shows that the non-dimensional force\displacement curves for the large and small pipes are virtually identical if the penetration velocities are scaled by the pipe diameters.

The effect of increasing the penetration speed by 33,000% from 0.0003V/D to 0.098V/D on the normalised force\displacement curves is shown in Figure D.5. The normalised force values for the normal and fast penetration speed tests are summarised in Table D.4. The overall shape of the curves after a depth of 0.1z/D are similar, however the maximum normalised force increases by 62% from 4.5 in the slow speed tests to 7.3 in the fast speed tests. This shows that the normalised penetration force is affected by rate, however to achieve significant changes in the normalised penetration force the rate needs to change by many orders of magnitude.

These tests indicate that further analysis can be conducted using a single normalised force\displacement curve, as it is representative of all of the force\displacement curves with the same normalised parameters. However the normalisation does not account for different soils.
### Table D.4 – Summary of Normalised Penetration Forces on Onsoy Clay

<table>
<thead>
<tr>
<th>$z/D$</th>
<th>Slow Penetration Test</th>
<th>Fast Penetration Test</th>
<th>Ratio of Fast to Slow Penetration Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0.00</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>0.1</td>
<td>2.81</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>0.2</td>
<td>3.51</td>
<td>5.92</td>
<td>1.67</td>
</tr>
<tr>
<td>0.3</td>
<td>3.90</td>
<td>6.53</td>
<td>1.67</td>
</tr>
<tr>
<td>0.4</td>
<td>4.23</td>
<td>6.95</td>
<td>1.67</td>
</tr>
<tr>
<td>0.5</td>
<td>4.51</td>
<td>7.30</td>
<td>1.62</td>
</tr>
<tr>
<td>Speed</td>
<td>0.0003V/D</td>
<td>0.098V/D</td>
<td>327</td>
</tr>
</tbody>
</table>

### Figure D.4 – Normalised Penetration Tests Using Small Diameter Pipe and Consistent Push in Speed
Skempton’s (1951) N values are compared to the penetration tests conducted on the Watchet Harbour and Onsoy clays, marine clays tested in the CARISIMA JIP. The non-dimensional backbone curves are shown graphically in Figure D.6 and tabulated in Table D.5. It can be seen that the Watchet Harbour normalised backbone curve compares well with both Skempton’s and Meyerhof’s normalised curves. The Onsoy normalised backbone curves have a similar shape to the published data, however the normalised curves with a penetration speed of 0.0003V/D have lower normalised force of 4.5 compared to Skempton’s 6.0 at 0.5z/D. The fast speed (0.098V/D) normalised penetration curve has a normalised force of 7.3 at 0.5z/D that is higher than Skempton’s values.

Skempton’s values of N are assumed to be conservative for use in soil stiffness models where the penetration speed is slow, such as in SCR installation from a J-Lay barge. However when the pipe is pushed into the soil, as in dynamic motion, the velocity of the pipe/soil interaction needs to be accounted.
Table D.5 – Comparison of Bearing Capacity Factors N

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>-</td>
<td>0.00</td>
</tr>
<tr>
<td>0.1</td>
<td>3.31</td>
<td>3.15</td>
<td>2.81</td>
<td>-</td>
<td>3.45</td>
</tr>
<tr>
<td>0.2</td>
<td>4.53</td>
<td>4.28</td>
<td>3.51</td>
<td>5.92</td>
<td>4.84</td>
</tr>
<tr>
<td>0.3</td>
<td>5.30</td>
<td>4.99</td>
<td>3.90</td>
<td>6.53</td>
<td>5.36</td>
</tr>
<tr>
<td>0.4</td>
<td>5.77</td>
<td>5.44</td>
<td>4.23</td>
<td>6.95</td>
<td>5.67</td>
</tr>
<tr>
<td>0.5</td>
<td>5.98</td>
<td>5.65</td>
<td>4.51</td>
<td>7.30</td>
<td>5.95</td>
</tr>
</tbody>
</table>

CARISIMA - Penetration Data
Comparison of Onsoy and Watchet Clays with Skempton (1951) and Meyerhof (1963)

Figure D.6 – Comparison of Penetration Data from Onsoy and Watchet Harbour Clays

D.5 Re-Penetration Test Data

The re-penetration test data consists of 4 pipe-soil interaction curves, which are the penetration curve, the unloading curve, the soil suction curve and the re-penetration curve. These are described in Chapter 7 and illustrated using CARISIMA II test data in Figure D.7. The test data is normalised using the method given in Chapter 7. A summary of the re-penetration test method is given below.
• The pipe is penetrated into the virgin soil at a speed of 0.0003V/D to a depth 0.5D.

• The penetration force is then reduced to 100N (normalised force 0.64) and the pipe is left for 12 hours or 48 hours for the 0.1016m and 0.2191m diameter pipes respectively.

• The pipe is then pulled out, clear of the trench that is formed by the penetration.

• The pipe is re-penetrated into the existing trench at a speed of 0.0003V/D to a depth of 1.0D or 1.5D.

The re-penetration tests show that as the pipe is pulled upwards from the trench the initial unloading curve is steep, indicating high soil stiffness. As the pipe moves further upwards a soil suction force is mobilised which resists the vertical motions of the pipe. After a distance of approximately 0.3D the soil suction bond breaks and the pipe moves clear of the trench. When the pipe is re-penetrated into the trench the bearing force starts at the depth of the soil suction breakout. The force then slowly increases to rejoin the backbone curve at a depth of approximately 1.0D.

These tests show the gradient of the pipe-soil interaction curves is steeper, and therefore the soil stiffness is greater on the unloading/soil suction curve than on the re-penetration curve.
CARISIMA II, Test 01
Pipe Diameter 0.1016m, Soil Shear Strength at 0.5D 3.8kPa

Figure D.7 – Re-penetration Curve

Figure D.8 – Normalised Re-penetration Curve
D.6 Vertical Pull Out Tests

The CARISIMA soil suction tests initially examined the effects of pull out velocity, consolidation time and pipe diameter. In addition a selection of tests were conducted with a 'vented' trench (a section of the trench wall was removed by hand to allow water to flow into the trench during the pull out test) or a 'remoulded' trench (using the same trench after a pull out test has been conducted). The test matrix summarising the tests from both phases of CARISIMA JIP is given in Table D.6. A summary of the pull out test results is given in Table D.8 and graphically in Figure D.9 and B.10.
## Table D.6 – CARISIMA Vertical Pull Out Test Matrix

<table>
<thead>
<tr>
<th>Consolidation Time (Hrs) and Pipe Size</th>
<th>Pull Out Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.005</td>
</tr>
<tr>
<td>12 (0.1016m D₀ Pipe)</td>
<td>4</td>
</tr>
<tr>
<td>48 (0.2191m D₀ Pipe)</td>
<td>-</td>
</tr>
<tr>
<td>0.167 (10min) (0.1016m D₀ Pipe)</td>
<td>-</td>
</tr>
<tr>
<td>1 (0.1016m D₀ Pipe)</td>
<td>-</td>
</tr>
<tr>
<td>2 (0.1016m D₀ Pipe)</td>
<td>2-1</td>
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<tr>
<td>18 (0.1016m D₀ Pipe)</td>
<td>-</td>
</tr>
<tr>
<td>42 (0.1016m D₀ Pipe)</td>
<td>6-2</td>
</tr>
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</table>

**Notes:**

- x CARISIMA I – tests on Onsoy Clay (top 2 rows)
- x-x CARISIMA II – tests on Watchet Harbour Clay
- x-x Pull out tests conducted with ‘venting’
- x-x Pull out tests conducted on remoulded clay
- x-x Cycling after ‘Push’ and after ‘Consolidation’
- x-x Cycling prior to pull out
- x-x Changing consolidation load, 3-1 – 50N, 7-1 – 150N.
- x-x Changing trench profile, 6-1 – 0.25z/D, 7-2 – 1.5w/D
Table D.7 – Summary of Pull Out Tests

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Diameter (m)</th>
<th>Undrained Soil Shear Strength (kPa)</th>
<th>Consolidation Time (hours)</th>
<th>Pull Out Speed (m/s)</th>
<th>Maximum Pull Out Force (N)</th>
<th>Break-out Disp (m)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cl, 1</td>
<td>0.1016</td>
<td>2.05</td>
<td>12</td>
<td>0.010</td>
<td>364.2</td>
<td>0.0292</td>
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<tr>
<td>Cl, 2</td>
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<td>12</td>
<td>0.080</td>
<td>603.0</td>
<td>0.0516</td>
<td></td>
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<tr>
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<td>12</td>
<td>0.010</td>
<td>465.1</td>
<td>0.0304</td>
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<td>2.55</td>
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<td>0.001</td>
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<td>0.0138</td>
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<tr>
<td>Cl, 5</td>
<td>0.1016</td>
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<td>12</td>
<td>0.079</td>
<td>643.4</td>
<td>0.0522</td>
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<td>12</td>
<td>0.010</td>
<td>291.2</td>
<td>0.0145 Vented</td>
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<tr>
<td>Cl, 7</td>
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<td>12</td>
<td>0.010</td>
<td>143.1</td>
<td>0.0245 Pre-cycling</td>
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<tr>
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<td>0.67</td>
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<td>133.3</td>
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<tr>
<td>Cl, 9</td>
<td>0.2191</td>
<td>2.40</td>
<td>48</td>
<td>0.022</td>
<td>2197.5</td>
<td>0.0796</td>
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<tr>
<td>Cl, 10</td>
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<td>48</td>
<td>0.172</td>
<td>3215.1</td>
<td>0.1332</td>
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<td>0.010</td>
<td>386.4</td>
<td>0.0335</td>
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<tr>
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<td>0.2191</td>
<td>2.55</td>
<td>48</td>
<td>0.022</td>
<td>2227.0</td>
<td>0.0584 Vented</td>
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<tr>
<td>Cl, 13</td>
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<td>2.02</td>
<td>12</td>
<td>0.010</td>
<td>328.9</td>
<td>0.0248 Vented</td>
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<td>0.81</td>
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<td>0.010</td>
<td>201.1</td>
<td>0.0427 Remoulded Clay</td>
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<td>2</td>
<td>0.010</td>
<td>501.8</td>
<td>0.0248</td>
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</tr>
<tr>
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<td>42</td>
<td>0.010</td>
<td>635.8</td>
<td>0.0280 Trench Depth 1.0D</td>
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</tr>
<tr>
<td>Cl, 2-1</td>
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<td>4.04</td>
<td>5</td>
<td>0.005</td>
<td>472.4</td>
<td>0.0231 Trench Depth 1.0D</td>
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<td>545.7</td>
<td>0.0276 Trench Depth 1.0D</td>
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<td>Cl, 3-1</td>
<td>0.1016</td>
<td>4.22</td>
<td>2</td>
<td>0.010</td>
<td>528.9</td>
<td>0.0250 Trench Depth 1.0D</td>
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<tr>
<td>Cl, 3-2</td>
<td>0.1016</td>
<td>4.1</td>
<td>1</td>
<td>0.010</td>
<td>655.7</td>
<td>0.0611 Trench Depth 1.0D</td>
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<tr>
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<td>3.95</td>
<td>0.167</td>
<td>0.010</td>
<td>472.4</td>
<td>0.0240 Trench Depth 1.0D</td>
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<tr>
<td>Cl, 4-2</td>
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<td>4.83</td>
<td>42</td>
<td>0.010</td>
<td>897.3</td>
<td>0.0600 Trench Depth 1.5D</td>
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<td>Cl, 5-1</td>
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<td>3.87</td>
<td>0.167</td>
<td>0.081</td>
<td>818</td>
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<tr>
<td>Cl, 5-2</td>
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<td>4.79</td>
<td>42</td>
<td>0.081</td>
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<td>3.49</td>
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<td>0.010</td>
<td>223.3</td>
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<td>4.69</td>
<td>42</td>
<td>0.005</td>
<td>659.1</td>
<td>0.0518 Trench Depth 1.5D</td>
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<tr>
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<td>2</td>
<td>0.010</td>
<td>620.8</td>
<td>0.0296 Trench Depth 1.5D</td>
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<td>5.2</td>
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<td>594.6</td>
<td>0.0175 Trench Depth 1.5D</td>
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<td>1</td>
<td>0.010</td>
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<td>4.61</td>
<td>42</td>
<td>0.010</td>
<td>247.3</td>
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<td>1</td>
<td>0.010</td>
<td>214.7</td>
<td>0.0085 Trench Depth 1.5D</td>
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<td>0.1016</td>
<td>5.06</td>
<td>42</td>
<td>0.010</td>
<td>673.5</td>
<td>0.0259 Trench Depth 1.5D</td>
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<td>0.167</td>
<td>0.081</td>
<td>937.9</td>
<td>0.0444 Trench Depth 1.5D</td>
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<td>4.65</td>
<td>18</td>
<td>0.010</td>
<td>558.3</td>
<td>0.0144 Trench Depth 1.5D</td>
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### Table D.8 – Summary of Normalised Parameters

<table>
<thead>
<tr>
<th>Test</th>
<th>Pull Out Velocity (D/s)</th>
<th>Maximum Pull Out Force (-)</th>
<th>Break Out Displacement (-)</th>
<th>Consolidation Parameter (N/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \frac{V}{D} )</td>
<td>( \frac{Q_s}{L D S_U} )</td>
<td>( \frac{\Delta_B}{D} )</td>
<td>( F_C \times \sqrt{\frac{c_{v f}}{D^2}} )</td>
</tr>
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<td>Cl, 1</td>
<td>0.0984</td>
<td>4.30</td>
<td>0.287</td>
<td>624</td>
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<tr>
<td>Cl, 2</td>
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<td>6.03</td>
<td>0.507</td>
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<td>4.49</td>
<td>0.299</td>
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<td>2.56</td>
<td>0.136</td>
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<td>0.514</td>
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<td>2.96</td>
<td>0.143</td>
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<td>6.58</td>
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<td>0.330</td>
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<td>3.94</td>
<td>0.244</td>
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<tr>
<td>Cl, 14</td>
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<td>6.04</td>
<td>0.420</td>
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<td>0.236</td>
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<td>5.98</td>
<td>0.862</td>
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<td>0.054</td>
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<td>2.77</td>
<td>0.172</td>
<td>738</td>
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<td>1.30</td>
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</table>
Figure D.9 – Summary of Normalised CARISIMA I Pull Out Curves

Figure D.10 – Summary of Normalised CARISIMA II Pull Out Curves
D.7 Cyclic Tests

The test matrix for the CARISIMA I and II cyclic tests are given in Table D.9 and B.10. A summary of the cyclic test with break out is given in able B.11. The analysis of these tests is detailed in Chapter 7.0.

Table D.9 – CARISIMA I, In Contact Cyclic Test Matrix

<table>
<thead>
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<th>Consolidation Time (Hrs) and Pipe Size</th>
<th>Cyclic Range of Contact Force (N)</th>
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<tr>
<td></td>
<td>0 to 100</td>
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<tr>
<td>12 (0.1016m D Pipe)</td>
<td>3 – 8, 13, 14</td>
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<td>48 (0.2191m D Pipe)</td>
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<td>67 (0.2191m D Pipe)</td>
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Table D.10 – CARISIMA II, In Contact Cycling Test Matrix

<table>
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<th>Consolidation Time (Hrs)</th>
<th>Cyclic Range of Contact Force (N)</th>
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<td></td>
<td>100 to -20</td>
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<tr>
<td>0</td>
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<td>0.5</td>
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<tr>
<td>39</td>
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Notes:

x-x Cycling conducted at trench depth of 0.25D
all tests use the small 0.1016m diameter pipe
### Table D.11 – CARISIMA II, Cyclic Tests with Break Out

<table>
<thead>
<tr>
<th>Consolidation Time (Seconds)</th>
<th>Pull Out Velocity (mm/s)</th>
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</thead>
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<tr>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td>5</td>
<td>-</td>
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<tr>
<td>30</td>
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<td>60</td>
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<tr>
<td>120 (2 min)</td>
<td>-</td>
</tr>
<tr>
<td>300 (5 min)</td>
<td>8-1d, 8-2d</td>
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</table>

**Notes:**
- x-x 50N Contact Force
- x-x 100N Contact Force
- x-x 150N Contact Force
- all tests use the small 0.1016m diameter pipe

### D.8 Reference

Marintek (2000) – Communications by the Author with the CARISIMA JIP.

Marintek (2000a) – “CARISIMA, Soil Parameters”, Report No. 700039.00.02, Trondheim, Norway

Marintek (2000b) – “CARISIMA, Interpretation of Suction Test Results”, Report No. 700039.00.03, Trondheim, Norway
APPENDIX E  LATERAL PIPE\SOIL INTERACTION

E.1  Introduction

This appendix presents the lateral pipe\soil interaction model developed by the writer using existing literature and the test data from the CARISIMA (Marintek, 2001A) and STRIDE JIPs (2H Offshore 2002).

An SCR may be considered as having two different forms of trench interaction as shown in Figure E.1.

- The portion of the riser length suspended in an open trench between the trench floor and the trench lip
- The portion of the riser length in an open trench resting on the trench floor

ROV footage indicates areas where the pipe may be buried within the trench due to sidewall collapse. However testing at Watchet Harbour indicated that the backfill material tends to be very soft and is not a significant contributor to interaction effects, so can be ignored for modelling purposes.

At position A-A' the riser is suspended between the trench floor and the trench lip. ROV footage of installed SCRs, given in Chapter 3, show that trench shapes in this part of the TDP have sloping walls. When the pipe is in the middle of the trench there are no vertical or lateral restraining forces on the pipe from the soil. When the pipe is pulled into contact with the trench wall the soil resistance forces and the sloping trench wall cause to pipe to climb upwards as well as being restrained laterally

ROV footage of installed SCRs, given in Chapter 3, and evidence from the STRIDE harbour tests indicates the riser will be in half diameter (D) close fitting trench within a larger box section trench, as shown in Figure E.1. As the pipe moves laterally towards the trench wall two forces are mobilised, a friction force and a passive soil resistance force. The pipe is assumed to have little vertical motion at this point. When the pipe is pulled into the main trench wall, the pipe will climb up the wall in a
similar manner to the previous case. The lateral soil resistance on the pipe will be higher than before due to the higher soil shear strength and the more vertical trench wall.

![Figure E.1 - Sketch of SCR TDP](image)

E.2 Test Method

The CARISIMA lateral testing series consists of two basic test types: vertically fixed tests (the pipe is fixed in the vertical axis) and vertically free tests (the pipe is free to move in the vertical axis). These tests are described in detail below.

In the vertically fixed tests the pipe was slowly penetrated to a shallow depth ($\approx 0.7D$). The vertical actuator was then locked at that displacement and a short lateral pull-out test was conducted, shown in Figure E.2. The pipe was then slowly penetrated to a slightly greater depth ($\approx 0.16D$) and another lateral pull-out test conducted in the same direction as the first lateral pull-out. This sequence was
continued for trench depths of \( \approx 0.25D, \approx 0.35D, \approx 0.45D, \approx 0.55D, \approx 0.7D, \approx 0.8D, \approx 0.9D, \approx 1.0D, \approx 1.5D, \approx 2.0D, \approx 2.5D \). The majority of the tests were conducted with a lateral pull-out velocity of 5mm/s, and a few tests conducted with a lateral pull-out velocity of 50mm/s.

In the vertically free tests the pipe was slowly penetrated to a depth, usually between 0.4D and 2.0D. The lateral pull-out was then conducted with the pipe free to move in the vertical axis. Tests were conducted at \( \approx 0.2D, \approx 0.4D, \approx 0.7D, \approx 1.0D, \approx 1.1D, \approx 1.5D, \approx 2.0D \) and \( \approx 2.5D \) penetration depths. The majority of the tests were conducted with a lateral pull-out velocity of 5mm/s with additional lateral tests conducted with pull-out velocities of 1mm/s and 50mm/s.

\[ \text{Vertically Fixed Tests} \]

\[ \text{Vertically Free Tests} \]

Figure E.2 – Example of Lateral Pull Out Tests

An example of the lateral pull-out soil resistance curve is given in Figure E.3. The lateral soil resistance force starts at the origin and then increases to the peak lateral force with a small increase in displacement. From the peak lateral force the force
gently decreases to a constant residual friction force as the displacement increases. The points of interest on the lateral pull-out curves that are used in the comparisons and the data analysis are:

- $P_{\text{max}}$, the peak lateral force
- $y_{p_{\text{max}}}$, the displacement at peak lateral force
- $Q_{p_{\text{max}}}$, the vertical force when the lateral force is at its peak
- $P_r$, the lateral residual force
- $Q_r$, the vertical residual force

![Graph of Lateral and Vertical Pipe/Soil Interaction Curves](image)

**Figure E.3 – Example Lateral and Vertical Pipe/Soil Interaction Curves**

**E.3 Lateral Pull Out Tests**

**E.3.1 Geotechnical Data**

Soil shear strength is measured using a T-Bar, which consists of a horizontal bar 20mm in diameter and 125mm long as shown in Figure E.4. The T-Bar was developed by the Norwegian Geotechnical Institute (NGI) specifically for the CARISIMA tests to measure shallow undrained shear strengths more reliably than a cone penetration test. The soil shear strengths are given in Table E.1.
Figure E.4 – Using the T-Bar on the Onsoy Clay Soils at NGI.
Photograph Courtesy of NGI (2002).
Table E.1 – Shear Strengths at Test Locations in Test Tank, Marintek (2001B)

<table>
<thead>
<tr>
<th>Lateral Test Number</th>
<th>Undrained Shear Strength at Surface (kPa)</th>
<th>Undrained Shear Strength Gradient (kPa/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H01</td>
<td>1.65</td>
<td>0.014</td>
</tr>
<tr>
<td>H02</td>
<td>2.0</td>
<td>0.0115</td>
</tr>
<tr>
<td>H03</td>
<td>1.95</td>
<td>0.0115</td>
</tr>
<tr>
<td>H04</td>
<td>1.95</td>
<td>0.013</td>
</tr>
<tr>
<td>H05</td>
<td>1.89</td>
<td>0.013</td>
</tr>
<tr>
<td>H06</td>
<td>1.6</td>
<td>0.014</td>
</tr>
<tr>
<td>H07</td>
<td>1.85</td>
<td>0.014</td>
</tr>
<tr>
<td>H08</td>
<td>1.4</td>
<td>0.012</td>
</tr>
<tr>
<td>H09</td>
<td>1.61</td>
<td>0.015</td>
</tr>
<tr>
<td>H10</td>
<td>1.56</td>
<td>0.01</td>
</tr>
<tr>
<td>H11</td>
<td>1.9</td>
<td>0.013</td>
</tr>
<tr>
<td>H12</td>
<td>1.5</td>
<td>0.01</td>
</tr>
<tr>
<td>H13</td>
<td>0.55</td>
<td>0.0025</td>
</tr>
<tr>
<td>H14</td>
<td>1.61</td>
<td>0.013</td>
</tr>
<tr>
<td>H15</td>
<td>1.65</td>
<td>0.015</td>
</tr>
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<td>H16</td>
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<tr>
<td>H17</td>
<td>1.2</td>
<td>0.014</td>
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</tbody>
</table>

E.3.2 Vertically Fixed Tests

The vertically fixed tests are described in Section E.2. A basic assessment of the test data is made using tests with a pull-out velocity of 5mm/s. The results are summarised in Table E.2 and Figure E.5.
Table E.2 – Summary of Vertically Fixed Test Data, Marintek (2001B)

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Lateral Pull-out Velocity (mm/s)</th>
<th>Trench Depth (h/D)</th>
<th>Lateral Force (P/[LDSu])</th>
<th>Vertical Force (Q/[LDSu])</th>
<th>Undrained Shear Strength (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H01A</td>
<td>5</td>
<td>0.07</td>
<td>0.95</td>
<td>1.77</td>
<td>1.75</td>
</tr>
<tr>
<td>H01B</td>
<td>5</td>
<td>0.16</td>
<td>1.36</td>
<td>2.07</td>
<td>1.88</td>
</tr>
<tr>
<td>H01C</td>
<td>5</td>
<td>0.25</td>
<td>1.64</td>
<td>2.08</td>
<td>2.01</td>
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<tr>
<td>H01D</td>
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<td>1.87</td>
<td>2.03</td>
<td>2.15</td>
</tr>
<tr>
<td>H01E</td>
<td>5</td>
<td>0.44</td>
<td>2.11</td>
<td>2.13</td>
<td>2.28</td>
</tr>
<tr>
<td>H01F</td>
<td>5</td>
<td>0.54</td>
<td>2.49</td>
<td>2.26</td>
<td>2.42</td>
</tr>
<tr>
<td>H01G</td>
<td>5</td>
<td>0.68</td>
<td>2.89</td>
<td>2.29</td>
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</tr>
<tr>
<td>H01H</td>
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<td>2.82</td>
</tr>
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<td>H01I</td>
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<td>0.97</td>
<td>3.79</td>
<td>2.43</td>
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</tr>
<tr>
<td>H08A</td>
<td>50</td>
<td>0.69</td>
<td>3.88</td>
<td>3.08</td>
<td>2.24</td>
</tr>
<tr>
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<td>50</td>
<td>1.10</td>
<td>6.08</td>
<td>3.43</td>
<td>2.74</td>
</tr>
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<td>H08C</td>
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<td>1.54</td>
<td>5.29</td>
<td>2.68</td>
<td>3.28</td>
</tr>
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<td>H08D</td>
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<td>2.01</td>
<td>6.54</td>
<td>2.71</td>
<td>3.85</td>
</tr>
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<td>H08E</td>
<td>5</td>
<td>2.48</td>
<td>7.92</td>
<td>2.73</td>
<td>4.42</td>
</tr>
</tbody>
</table>

CARISIMA Vertically Fixed Lateral Pull-out Tests
Lateral and Vertical Forces with Penetration Depth

Figure E.5 – Summary of Vertical and Lateral Forces with Penetration Depth
### E.3.3 Vertically Free Tests

The vertically free tests are described in Section E.2. The results are summarised in Table E.3 and in Figures E.6 to E.10. For this analysis only the tests with a lateral pull-out velocity of 5mm/s are considered, tests H07b (1mm/s) and H05b (50mm/s) are not used.

**Table E.3 – Summary of Vertically Free Lateral Tests, Marintek (2001B)**

<table>
<thead>
<tr>
<th>Test No</th>
<th>Trench Depth</th>
<th>Pull Speed</th>
<th>Shear Strength</th>
<th>Max Lateral Force</th>
<th>Displacement of Peak Lateral Force</th>
<th>Vertical Force</th>
<th>Climb Angle</th>
<th>Residual Lateral Force</th>
<th>Residual Vertical Force</th>
</tr>
</thead>
<tbody>
<tr>
<td>H02A</td>
<td>0.22</td>
<td>5</td>
<td>2.26</td>
<td>0.96</td>
<td>0.39</td>
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<td>9.9</td>
<td>0.46</td>
<td>1.20</td>
</tr>
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<td>H02B</td>
<td>0.41</td>
<td>5</td>
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<td>1.39</td>
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<td>1.11</td>
<td>14.3</td>
<td>0.49</td>
<td>1.20</td>
</tr>
<tr>
<td>H15A</td>
<td>0.41</td>
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<td>2.27</td>
<td>1.27</td>
<td>0.62</td>
<td>0.75</td>
<td>25.1</td>
<td>0.24</td>
<td>0.87</td>
</tr>
<tr>
<td>H17A</td>
<td>0.42</td>
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<td>1.80</td>
<td>1.21</td>
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<td>0.42</td>
<td>34.6</td>
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<tr>
<td>H14A</td>
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<td>27.1</td>
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<td>1.76</td>
<td>0.76</td>
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<tr>
<td>H12A</td>
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<td>34.7</td>
<td>0.47</td>
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</tr>
<tr>
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<td>-</td>
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<tr>
<td>H09A</td>
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<td>3.30</td>
<td>3.86</td>
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<td>1.53</td>
<td>20.9</td>
<td>0.97</td>
<td>2.06</td>
</tr>
<tr>
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</tr>
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<td>-</td>
</tr>
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</tr>
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<td>H03B</td>
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<td>3.71</td>
<td>3.30</td>
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<td>0.91</td>
<td>23.7</td>
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<td>-</td>
</tr>
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<td>4.82</td>
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<td>1.48</td>
<td>20.6</td>
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<td>-</td>
</tr>
<tr>
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<td>5.25</td>
<td>1.50</td>
<td>0.57</td>
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<td>-</td>
<td>-</td>
</tr>
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<td>-</td>
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<td>H08F</td>
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<td>4.42</td>
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<td>1.60</td>
<td>0.69</td>
<td>16.7</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
CARISIMA I - Lateral Test Data
Lateral Force Versus Trench Depth in Free Pulls

\[ y = 2.3261x + 0.3133 \]
\[ R^2 = 0.8211 \]

Figure E.6 – Peak lateral Force with Trench Depth

CARISIMA I - Lateral Test Data
Position of Peak Lateral Force Versus Trench Depth,
Free Pulls at 5mm/s Pull Out Velocity

\[ y = 0.551x + 0.3728 \]
\[ R^2 = 0.8712 \]

Figure E.7 – Position of the Peak lateral Force with Trench Depth
CARISIMA I - Lateral Test Data
Residual Lateral and Vertical Forces with Trench Depth, 5mm/s Pull Out Velocity

Figure E.8 – Residual Force with Trench Depth

CARISIMA I - Lateral Test Data
Residual Lateral and Vertical Forces, 5mm/s Pull Out Velocity

$y = 0.457x$

$R^2 = 0.8913$

Figure E.9 – Relationship between Lateral and Vertical Residual Forces
E.4 Experimental Data And Model Development

The lateral pipe/soil interaction model developed is based on the CARISIMA free pull tests and observations from the STRIDE JIP harbour tests. Two lateral pipe/soil interaction models are developed based on the test data from the two JIPs; one with the riser suspended between the trench floor and the trench lip, the second with the riser supported by the trench floor.

An example of the lateral pipe/soil interaction model is given in Figure E.9. The soil resistance force starts at the origin and then increases to the peak lateral force with a small increase in displacement. From the peak lateral force the force gently decreases to a constant residual friction force as the displacement increases.

E.5 Riser Suspended in Trench

An example of a riser moving laterally in a trench when the riser is suspended is shown in Figure E.10. From the centre line of the trench the riser moves laterally with no resisting soil forces. As the riser contacts the trench wall the initial soil restraining force mobilises. The pipe continues to move into the trench wall and the
restraining force continues to increase until the peak lateral force is reached, \( P_{\text{max}} \), and the clay yields. The lateral restraining force then drops to the residual friction force as the riser slides out of the trench and onto the seabed.

![Lateral Soil Model for Section A-A'](image)

**Figure E.11 - Lateral Soil Model for Section A-A’**

**E.6 Riser on Trench Floor**

An example of a riser moving laterally in a trench when the riser is lying on the trench floor is shown in Figure E.11. From the centre line of the trench the riser moves laterally with a friction force applied. The riser then contacts the trench wall and the restraining force increases until a maximum peak lateral force is reached, and the clay yields. The riser will push through the trench wall to some extent and climb up out of the trench. The lateral restraining force then drops to the residual (or friction) force as the riser slides out of the trench and onto the seabed.
The assumptions for the models based on the CARISIMA test data are:

- No direct account of the vertical force is made
- The peak lateral force, \( P_{\text{MAX}} \) increases with trench depth, Figure E.5
- The displacement at the peak lateral force, \( y_{p_{\text{max}}} \) increases with increasing trench depth, Figure E.6
- The residual friction force does not change with trench depth, Figure E.7
- The residual lateral force is related to the residual vertical force by a friction coefficient, Figure E.8
- The shape of the curve is from the trend line of the normalised free pull test data, Figure E.3, Figure E.10 and Figure E.11
The assumptions for the models based on the STRIDE test data are:

- The majority of lateral motion will occur during riser pull-out or slow drift
- Environmental loading causes small lateral movements at the TDP. The loading and unloading force/displacement curves are assumed to be similar for small motions and for simplicity follow the lateral pipe/soil interaction model

E.7 Shape of the Pipe/Soil Interaction Model

The shape of the lateral pipe/soil interaction model is determined by comparing normalised pipe/soil lateral interaction curves for each test. The lateral force is divided by the peak lateral force and the displacement is divided by the displacement at the peak lateral force. This creates a lateral model with a peak lateral force of one and a displacement at the peak lateral force of one as shown in Figure E.5. From this data the shape of all the lateral soil curves can be approximated as shown in Figure E.12, Figure E.13 and Table E.4. The maximum and minimum envelopes of the test data are also included to show the spread of the normalised test data.
Table E.4 – Normalised Lateral Pipe/Soil Interaction Model

<table>
<thead>
<tr>
<th>Normalised Lateral Pipe/Soil Interaction Model</th>
<th>Lower Lateral Soil Model Envelope</th>
<th>Upper Lateral Soil Model Envelope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normalised Displacement (-)</td>
<td>Normalised Force (-)</td>
<td>Normalised Force (-)</td>
</tr>
<tr>
<td>0.0</td>
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<td>0.00</td>
</tr>
<tr>
<td>0.2</td>
<td>0.15</td>
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<td>5.0</td>
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<td>0.10</td>
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</table>
Figure E.13 – Normalised Force and Displacement Test Data

Figure E.14 – Lateral Pipe/Soil Interaction Model
E.8 Derived Relationships

Using the data from the CARISIMA tests the following equations are derived for the lateral pipe/soil interaction model.

Peak lateral force:

\[ P_{\text{max}} = \left( 2.33 \frac{z}{D} + 0.31 \right) S_u D \]  \hspace{1cm} (E.1)

where

- \( P_{\text{max}} \) is the maximum lateral force
- \( z \) is the trench depth
- \( D \) is the riser diameter
- \( S_u \) is the undrained shear strength at invert level

Displacement at peak lateral force:

\[ y_{P_{\text{max}}} = \left( 0.55 \frac{z}{D} + 0.37 \right) D \]  \hspace{1cm} (E.2)

where

- \( y_{P_{\text{max}}} \) is the displacement at the peak lateral force

Residual force:

\[ P_R = 0.46 Q_R \]  \hspace{1cm} (E.3)

where

- \( P_R \) is the lateral residual force
- \( Q_R \) is the vertical residual force, which may be calculated from the submerged pipe weight.

E.9 Conclusions

Analysis is conducted using information from the STRIDE and CARISIMA JIPs to determine a lateral pipe/soil interaction model to be used for analysis of SCRs. Two trench models are developed; the first with the riser suspended between the trench floor and the trench lip. The second with the riser supported by the trench floor. Further modelling assumptions are made using data from the 2 JIPs.
The limitations of this lateral pipe/soil interaction model are that it is applicable for large quasi-static motions representing slow drift or riser pull-out loading and small dynamic TDP motions. However the model does not accommodate large dynamic TDP motions (greater than approximately two diameters).

E.10 Reference


APPENDIX F  PUBLICATIONS ASSOCIATED WITH THIS THESIS

This appendix presents copies of STRIDE JIP, conference and journal papers written by the writer as a result of this work.


1.0 INTRODUCTION

Part of the additional scope of work for STRIDE Phase 4 includes investigation into the pull-out resistance of a 6-5/8" OD pipe in a clay soil. The STRIDE and CARISIMA JIPs both conducted tests examining lateral riser/soil interaction. The STRIDE tests consisted of a 328ft long, 5-5/8" OD test riser draped into a harbour which examined the interaction of a pipe with a very soft natural clay of high plasticity. The CARISIMA tests were performed on a 16" long, 4" OD pipe section in a test tank to investigate the 2D interaction of the test pipe with a very soft reconstituted medium plastic clay. Together the results of the STRIDE and CARISIMA JIPs provide a basis for modelling the lateral interaction of riser pipe with very soft highly plastic clays.

This technical note describes the assumptions made and the methods used to develop the lateral pipe/soil interaction model.

2.0 SUMMARY

Analysis is conducted using information from the STRIDE and CARISIMA JIPs to determine a lateral pipe/soil interaction model to be used for analysis of SCRs. Two trench models are developed: The first with the riser suspended between the trench floor and the trench lip, Figure 3.2. The second with the riser supported by the trench floor, Figure 3.3. Further modelling assumptions are made using data from the 2 JIPs.

An example of the lateral pipe/soil interaction model is given in Figure 4.1. The soil resistance force starts at the origin and then increases to the peak lateral force with a small increase in displacement. From the peak lateral force the force gently decreases to a constant residual friction force as the displacement increases.

Lateral pipe/soil interaction models are produced for 12" nominal OD SCR's in the Gulf of Mexico and West Africa. Each lateral pipe/soil interaction model consists of 5 curves representing 0.5D, 1D, 2D, 3D and 4D penetration depths within the 4D deep trench. The trench shape used (4D deep by 3D wide) is typical of those seen in the ROV footage of SCR trenches.

The limitations of this lateral pipe/soil interaction model are that it is applicable for large quasi-static motions representing slow drift or riser pull-out loading and small dynamic TDP motions. However the model does not accommodate large dynamic TDP motions (greater than approximately 2D).
3.0 PROBLEM DESCRIPTION

Observations from a number of deepwater GoM platforms show that SCR’s supported on soft clay seabeds lie in relatively wide trenches many diameters deep. These trenches are formed soon after installation and generally remain open, although some localised sidewall collapses do occur.

As shown in Figure 3.1 an SCR may be considered as having two different forms of trench interaction.

- The portion of the riser length suspended in an open trench between the trench floor and the trench lip, Figure 3.2
- The portion of the riser length in an open trench resting on the trench floor, Figure 3.3

ROV footage has also indicated areas where the pipe may be buried within the trench due to sidewall collapse. However testing at Watchet Harbour indicated that the backfill material tends to be very soft and is not a significant contributor to interaction effects, so can be ignored for modelling purposes.

![Figure 3.1 – Sketch of SCR TDP](image)

3.1 Riser Suspended in Trench

At position A-A’ the riser is suspended between the trench floor and the trench lip. ROV footage of installed SCRs show that trench shapes in this part of the TDP have sloping walls. When the pipe is in the middle of the trench there are no vertical or lateral restraining forces on the pipe from the soil. When the pipe is pulled into contact with the trench wall the soil resistance forces and the sloping trench wall cause to pipe to climb upwards as well as being restrained laterally, Figure 3.2.
3.2 Riser on Trench Floor

ROV footage of installed SCRs and evidence from the STRIDE harbour tests indicates the riser will be in $\frac{1}{2}D$ close fitting trench within a larger box section trench, Figure 3.3. As the pipe moves laterally towards the trench wall two forces are mobilised, a friction force and a passive soil resistance force. The pipe is assumed to have little vertical motion at this point. When the pipe is pulled into the main trench wall, the pipe will climb up the wall in a similar manner to the previous case. The lateral soil resistance on the pipe will be higher than before due to the higher soil shear strength and the more vertical trench wall.

4.0 EXPERIMENTAL DATA AND MODEL DEVELOPMENT

The lateral pipe/soil interaction model developed is based on the CARISIMA free pull tests [2] and observations from the STRIDE JIP harbour tests [1]. Two lateral pipe/soil interaction models are developed based on the test data from the two JIPs; one with the riser suspended between the trench floor and the trench lip, the second with the riser supported by the trench floor.
An example of the lateral pipe/soil interaction model is given in Figure 4.1. The soil resistance force starts at the origin and then increases to the peak lateral force with a small increase in displacement. From the peak lateral force the force gently decreases to a constant residual friction force as the displacement increases.

Figure 4.1 – Lateral Pipe/Soil Interaction Model

4.1 Riser Suspended in Trench

Figure 4.2 shows an example of a riser moving laterally in a trench when the riser is suspended. From the centre line of the trench the riser moves laterally with no resisting soil forces. As the riser contacts the trench wall the initial soil restraining force mobilises. The pipe continues to move into the trench wall and the restraining force continues to increase until the peak lateral force is reached, $P_{\text{max}}$, and the clay yields. The lateral restraining force then drops to the residual lateral force as the riser slides out of the trench and onto the seabed.
4.2 Riser on Trench Floor

Figure 4.3 shows an example of a riser moving laterally in a trench when the riser is lying on the trench floor. From the centre line of the trench the riser moves laterally, opposed by a soil resistance force. The riser then contacts the trench wall and the restraining force increases until a maximum peak lateral force is reached, and the clay yields. The riser will push through the trench wall to some extent and climb up out of the trench. The lateral restraining force then drops to the residual lateral force as the riser slides out of the trench and onto the seabed.
The assumptions for the models based on the CARISIMA test data are:

- No direct account of the vertical force is made
- The peak lateral force, $P_{\text{MAX}}$ increases with trench depth, Figure A1.1
- The displacement at the peak lateral force, $Y_{\text{pmax}}$ increases with increasing trench depth, Figure A1.2
- The residual friction force does not change with trench depth, Figure A1.3
- The residual lateral force is related to the residual vertical force by a friction coefficient, Figure A1.4
- The shape of the curve is from the trend line of the normalised free pull test data, Figure A1.5

The assumptions for the models based on the STRIDE test data are:

- The majority of lateral motion will occur during riser pull-out or slow drift
- Environmental loading causes small lateral movements at the TDP. The loading and unloading force/displacement curves are assumed to be similar for small motions and for simplicity follow the lateral pipe/soil interaction model
4.3 Shape of the Pipe/Soil Interaction Model

The shape of the lateral pipe/soil interaction model is determined by comparing normalised pipe/soil lateral interaction curves for each test. The lateral force is divided by the peak lateral force and the displacement is divided by the displacement at the peak lateral force. This creates a lateral loading model with a peak lateral force of 1.0 and a displacement at the peak lateral force of 1.0, Figure A1.5. From this data the shape of all the lateral soil curves can be approximated, Figure 4.4 and Table 4.1. The maximum and minimum envelopes of the test data are also included to show the spread of the normalised test data.

<table>
<thead>
<tr>
<th>Normalised Lateral Pipe/Soil Interaction Model</th>
<th>Lower Lateral Soil Model Envelope</th>
<th>Upper Lateral Soil Model Envelope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normalised Displacement (-)</td>
<td>Normalised Force (-)</td>
<td>Normalised Force (-)</td>
</tr>
<tr>
<td>0.0</td>
<td>0.02</td>
<td>0.00</td>
</tr>
<tr>
<td>0.2</td>
<td>0.15</td>
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</tr>
<tr>
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</tr>
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</tr>
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<td>0.93</td>
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<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>1.1</td>
<td>1.00</td>
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<tr>
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<td>0.48</td>
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</tr>
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</tr>
<tr>
<td>5.0</td>
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<td>0.25</td>
</tr>
</tbody>
</table>

Table 4.1 – Normalised Lateral Pipe/Soil Interaction Model
5.0 CASE STUDY SCR SOIL INTERACTION CURVES

Lateral pipe/soil interaction models are given for both the Gulf of Mexico and West Africa SCR’s. Each lateral pipe/soil interaction models consists of 5 curves representing 0.5D, 1D, 2D, 3D and 4D depths within the 4D deep trench.

5.1 Assumptions

The development of the lateral pipe/soil interaction models for the case study risers uses the soil and trench properties given in Table 5.1 and the riser properties given in Table 5.2. It should be noted that the assumed trench shape (3D wide by 4D deep) is typical of those seen from ROV footage and this is considered a realistic approach for the analysis.
## Table 5.1 – Soil Properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Gulf of Mexico SCR</th>
<th>West Africa SCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface Undrained Shear Strength</td>
<td>2.63 kPa</td>
<td>1.92 kPa</td>
</tr>
<tr>
<td>Undrained Shear Strength Gradient</td>
<td>1.26 kPa/m</td>
<td>1.19 kPa/m</td>
</tr>
<tr>
<td>Submerged Unit Weight of Soil</td>
<td>4.4 kN/m³</td>
<td>3.0 kN/m³</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>50%</td>
<td>90%</td>
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<tr>
<td>Sensitivity of the Clay</td>
<td>3.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Assumed Trench Width</td>
<td>3.0D</td>
<td>3.0D</td>
</tr>
<tr>
<td>Assumed Trench Depth</td>
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<td>4.0D</td>
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</table>

## Table 5.2 – SCR Properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Gulf of Mexico SCR</th>
<th>West Africa SCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outer Diameter</td>
<td>0.324m (12¾&quot;)</td>
<td>0.324m (12¾&quot;)</td>
</tr>
<tr>
<td>Wall Thickness</td>
<td>0.0205m (0.8&quot;)</td>
<td>0.0188m (0.7&quot;)</td>
</tr>
<tr>
<td>Coating Thickness</td>
<td>50mm (2.0&quot;)</td>
<td>50mm (2.0&quot;)</td>
</tr>
<tr>
<td>Steel Density</td>
<td>7850 kg/m³ (841lb/ft³)</td>
<td>7850 kg/m³ (841lb/ft³)</td>
</tr>
<tr>
<td>External Coating Density</td>
<td>700 kg/m³ (75lb/ft³)</td>
<td>700 kg/m³ (75lb/ft³)</td>
</tr>
<tr>
<td>Internal Fluid Density</td>
<td>800 kg/m³ (86lb/ft³)</td>
<td>800 kg/m³ (86lb/ft³)</td>
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<tr>
<td>In Service Weight in Water</td>
<td>100 kg/m (67lb/ft)</td>
<td>89 kg/m (60lb/ft)</td>
</tr>
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</table>
5.2 Gulf of Mexico SCR

The lateral pipe/soil interaction model for the Gulf of Mexico SCR at 0.5D, 1D, 2D, 3D and 4D depths in a 3D wide by 4D deep trench are given in Table 5.3 and Figure 5.1.

<table>
<thead>
<tr>
<th>0.5D Depth</th>
<th>1D Depth</th>
<th>2D Depth</th>
<th>3D Depth</th>
<th>4D Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Disp (m)</td>
<td>Force (kN)</td>
<td>Disp (m)</td>
<td>Force (kN)</td>
<td>Disp (m)</td>
</tr>
<tr>
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<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
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<tr>
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<td>0.581</td>
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<td>8.251</td>
<td>0.452</td>
<td>12.915</td>
</tr>
</tbody>
</table>

Table 5.3 – Lateral Soil Curve for 24” Oil Export Riser

Figure 5.1 - Lateral Soil Curve for Gulf of Mexico SCR
5.3 West Africa SCR

The lateral pipe/soil interaction model for the West Africa SCR at 0.5D, 1D, 2D, 3D and 4D depths in a 3D wide by 4D deep trench are given in Table 5.4 and Figure 5.2.

<table>
<thead>
<tr>
<th>0.5D Depth</th>
<th>1D Depth</th>
<th>2D Depth</th>
<th>3D Depth</th>
<th>4D Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Disp (m)</td>
<td>Force (kN)</td>
<td>Disp (m)</td>
<td>Force (kN)</td>
<td>Disp (m)</td>
</tr>
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<tr>
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<tr>
<td>2.92</td>
<td>0.404</td>
<td>1.989</td>
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<td>3.55</td>
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<td>3.547</td>
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<tr>
<td>12.92</td>
<td>0.404</td>
<td>8.251</td>
<td>0.404</td>
<td>12.915</td>
</tr>
</tbody>
</table>

Table 5.3 – Lateral Soil Curve for 24” Oil Export Riser

Figure 5.2 - Lateral Soil Curve for West Africa SCR
6.0 REFERENCES


A1.0 REFERENCE DATA

The CARISIMA free pull tests are used to define the basic relationships that are used in the development of the lateral pipe/soil interaction models. The test data showing a relationship between the peak lateral force and trench depth is given in Figure A1.1. It is found that a linear trend line gives the best fit with a correlation coefficient of 0.82. The free pull test data has been non-dimensionalised using the following schemes:

- Peak lateral force: \( \frac{P_{\text{max}}}{L \times D \times S_U} \)
- Trench depth: \( \frac{h}{D} \)

Where

- \( P_{\text{max}} \) Peak lateral force, (N)
- \( L \) Length of test pipe, (m)
- \( D \) Diameter of test pipe, (m)
- \( S_U \) Undrained shear strength of the soil, (Pa)
- \( h \) Trench depth, (m)

![Figure A1.1 - Peak lateral Force with Trench Depth](image)
The test data showing a relationship between the displacement at the peak lateral force and trench depth is given in Figure A1.2. It is found that a linear trend line gives the best fit with a correlation coefficient of 0.87. The free pull test data has been non-dimensionalised using the following schemes:

- Position of peak lateral force: \( \frac{y_{p_{\text{max}}}}{D} \)
- Trench depth: \( \frac{h}{D} \)

Where

- \( y_{p_{\text{max}}} \) Displacement at peak lateral force, (m)
- \( D \) Diameter of test pipe, (m)
- \( h \) Trench depth, (m)

![Figure A1.2 - Position of the Peak lateral Force with Trench Depth](image-url)

\[
y = 0.561x + 0.3728 \\
R^2 = 0.8712
\]
Figure A1.1 shows the scatter of results between the residual forces and the trench depth. As expected it is found that there is no trend between the residual forces and the trench depth. The free pull test data has been non-dimensionalised using the following schemes:

- Residual force: \( \frac{F_r}{L \times D \times S_U} \)
- Trench depth: \( \frac{h}{D} \)

Where

- \( F_r \) Residual force, (N)
- \( L \) Length of test pipe, (m)
- \( D \) Diameter of test pipe, (m)
- \( S_U \) Undrained shear strength of the soil, (Pa)
- \( h \) Trench depth, (m)

![Graph showing Residual Force with Trench Depth](image-url)

Figure A1.3 – Residual Force with Trench Depth
The test data showing a relationship between the residual lateral force and the residual vertical force is given in Figure A1.4. It is found that a linear trend line gives the best fit with a correlation coefficient of 0.89. The free pull test data has been non-dimensionalised using the following schemes:

- Residual lateral force: \[
\frac{P_r}{L \times D \times S_U}
\]
- Residual vertical force: \[
\frac{Q_r}{L \times D \times S_U}
\]

Where

- \(P_r\) Residual lateral force, (N)
- \(Q_r\) Residual vertical force, (N)
- \(L\) Length of test pipe, (m)
- \(D\) Diameter of test pipe, (m)
- \(S_U\) Undrained shear strength of the soil, (Pa)

Figure A1.4 – Relationship between Lateral and Vertical Residual Forces
Figure A1.5 shows the CARISIMA lateral free pull curves normalised using the following methods:

- Lateral force is divided by the peak lateral force
- Displacement is divided by the position of the peak lateral force

This produces a series of curves with a peak lateral force of 1.0 and a position of the peak lateral force of 1.0. Three trend lines have been plotted with the normalised data, the minimum and maximum envelope of the normalised data and the best fit curve.

Figure A1.5 – Normalised Force and Displacement Test Data
1.0 INTRODUCTION

Part of the scope of work for STRIDE Phase 4 includes investigation into the pull-out resistance of a pipe in a clay soil. Experiments are conducted which look at the effects of pull-out rate, pipe diameter, consolidation time and consolidation load [1]. The results of this work are used to develop a model for analysing the effect of soil suction on steel catenary risers. This model has been used in ABAQUS and ANSYS software and may be implemented in other general purpose FEA programs that have contact elements with a suction modelling capability. The objectives of this technical note is to define the analytical model in order that it can be used by others.

2.0 SOIL SUCTION MODEL

The soil suction model is based on experimental data observed during STRIDE and CARISIMA testing, Appendix 1.0. For analysis purposes this is modelled in 3 linear phases as shown in Figure 2.1.

- Suction mobilisation – As the pipe initially moves upwards the suction force increases from zero to the maximum value
- Suction plateau – The suction force remains constant as the pipe moves further upwards
- Suction release – Under further upward movement the suction force reduces from its maximum to zero at the break-out displacement

The soil suction model has two defined limits the maximum uplift resistance force and the break-out displacement from which all points on the soil suction model are derived. The values to be used for each of these parameters, defined below, depends on the type of SCR analysis to be conducted.
2.1 Estimation of Maximum Soil Suction Force

Maximum soil suction force is estimated using the formulae below [1]:

\[ Q_{S,\text{MAX}} = k_c \times k_v \times k_t \times N \times L \times D \times S_u \]  
\[ k_v = k_F \left( \frac{V}{D} \right)^{n_F} \]  
\[ k_t = k_{TF} \frac{F_C \sqrt{C_t t}}{L D^2} + C_{TF} \]  

Where:
- \( Q_{S,\text{MAX}} \) is the maximum uplift force (kN)
- \( k_c \) is the cyclic loading factor (no units)
- \( k_v \) is an empirical pull-out velocity factor (no units)
- \( k_t \) is a consolidation time factor (no units)
- \( N \) is the bearing capacity factor, \( N = 5.92 \) at 0.5D penetration (no units)
- \( L \) is the length of the pipe (m)
- \( D \) is the outer diameter of the pipe (m)
- \( S_u \) is the undrained soil shear strength (kPa)
- \( k_F \) is an empirically derived constant from CARISIMA and STRIDE test data \((s_{s'})\)
V: pull-out velocity (m/s)

\( n_F \): an empirically derived constant from CARISIMA and STRIDE test data (no units)

\( k_{TF} \): an empirically derived constant from CARISIMA II and STRIDE test data, \( k_{TF} = 0.00033 \, (m^2/N) \)

\( F_c \): consolidation force (N)

\( c_v \): coefficient of consolidation (m\(^2\)/year)

\( t \): consolidation time (years)

\( C_{TF} \): an empirically derived constant from CARISIMA II and STRIDE test data, \( C_{TF} = 0.9 \) (no units)

When using the formula the following assumptions are made:

- The factors \( k_F, k_c \) and \( n_F \) are based on CARISIMA and STRIDE data and may vary with soil type. Recommended values are given in Tables 2.1 and 2.2.
- The factor \( N \) is the standard bearing capacity factors, and is equal to 5.92 for a pipe penetration depth of 0.5D [3]
- Extreme storm and first order fatigue analysis have a reduced soil suction force due to the following:
  - Short Consolidation Times. The maximum suction force for a pull-out after a short consolidation time can be estimated by using the remoulded undrained shear strength as opposed to the undisturbed undrained shear strength. A short consolidation time is analogous to the rest time between dynamic TDP motions of an SCR
  - Cyclic Loading. Repeated pull-out in a remoulded trench causes the soil to liquefy and the maximum dynamic soil suction force to drop as shown in Appendix A2.0
- The undrained shear strength to be used is the (undisturbed or remoulded) undrained shear strength at the assumed trench depth
- The effect of pipe peeling is assumed to be negligible
- Plasticity index is accounted for using the factors in Table 2.1

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Clay Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Onsoy</td>
</tr>
<tr>
<td>Plasticity Index (%)</td>
<td>30</td>
</tr>
<tr>
<td>( k_F )</td>
<td>1.12</td>
</tr>
<tr>
<td>( n_F )</td>
<td>0.18</td>
</tr>
</tbody>
</table>

Table 2.1 – Empirical Factors for Maximum Uplift Force
Soil Suction Model for Analysis Purposes

<table>
<thead>
<tr>
<th>SCR Motion</th>
<th>Undrained Shear Strength, $S_u$</th>
<th>Cyclic Loading Factor, $k_C$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slow Drift</td>
<td>Undisturbed</td>
<td>1.0</td>
</tr>
<tr>
<td>Extreme Storm</td>
<td>Remoulded</td>
<td>0.56</td>
</tr>
<tr>
<td>First Order Fatigue</td>
<td>Remoulded</td>
<td>0.56</td>
</tr>
</tbody>
</table>

Table 2.2 – Shear Strengths and Cyclic Loading Factors from Test Data

2.2 Estimation of Break-out Displacement

Break-out displacement is estimated using the formulae below [1]:

$$\Delta_B = k_{DV} \times k_{DT} \times D \quad (2.4)$$

$$k_{DV} = k_D \times V^{n_D} \quad (2.5)$$

$$k_{DT} = k_{DTF} \frac{F_c \sqrt{c_T}}{L D^2} + C_{DTF} \quad (2.6)$$

Where:

- $\Delta_B$: break-out displacement (m)
- $k_{DV}$: an empirical break-out displacement factor (no units)
- $k_{DT}$: consolidation time factor (no units)
- $k_D$: an empirically derived constant from CARISIMA test data, ($[m/s]^{n_D}$)
- $n_D$: an empirically derived constant from CARISIMA test data (no units)
- $k_{DTF}$: an empirically derived constant from CARISIMA II and STRIDE test data, $k_{TF} = 0.0009 \ (m^2/N)$
- $C_{DTF}$: an empirically derived constant from CARISIMA II and STRIDE test data, $C_{DTF} = 0.8$ (no units)

When using the formula the following assumptions are made:

- The factors $k_D$ and $n_D$ are based on CARISIMA data and vary with soil type
- Break-out displacement is independent of trench depth
- The suction mobilisation distance is proportional to the break-out displacement
- The plateau distance is proportional to the break-out displacement
- The effect of pipe peeling is assumed to be negligible
- Plasticity index is accounted for using the factors in Table 2.3
### Table 2.3 – Empirical Factors for Break Out Displacement

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Clay Type</th>
<th>Onsy</th>
<th>Watchet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plasticity Index (%)</td>
<td>30</td>
<td>42</td>
<td></td>
</tr>
<tr>
<td>$k_D$</td>
<td>0.98</td>
<td>0.83</td>
<td></td>
</tr>
<tr>
<td>$n_D$</td>
<td>0.26</td>
<td>0.19</td>
<td></td>
</tr>
</tbody>
</table>

### 3.0 REFERENCES


A1.0 NORMALISED TEST DATA

Figure A1.1 [1] shows the CARISIMA soil suction curves normalised using the following methods:

- Uplift resistance force is divided by the maximum uplift force
- Displacement is divided by the break-out displacement

This produces a series of curves with a maximum uplift resistance force of 1.0 and a break-out displacement of 1.0.

Figure A1.1 – Normalised Force and Displacement Test Data
A2.0 CYCLIC LOADING DATA

Figure A2.1 [1] shows the effect of cyclic loading on the suction force of a pipe in a remoulded trench. After the third cycle the suction force is about 56% of the initial remoulded soil suction force.

Figure A2.1 – Effect of Cyclic Loading from CARISIMA Test a07
A3.0 COMPARISON BETWEEN EXPERIMENTAL DATA AND STRIDE MODEL

Figure A3.1 shows a comparison between the experimental data and calculated data using the STRIDE soil suction model. The maximum pull out force from CARISIMA phases I and II is plotted in the horizontal axis against the calculated data from the STRIDE soil suction model, equations (2.1), (2.2) and (2.3) in the vertical axis. The calculation uses the pipe diameter, pipe length, undrained shear strength, pull-out velocity, consolidation time and consolidation load from CARISIMA reports [2]. The green line in Figure A3.1 represents an exact correlation between experimental and calculated data, and any distance between the points and the line represent some error. As can be seen from Figure A3.1 the correlation factor, R, between the STRIDE model and the experimental data is 0.98, which shows a good correlation between the STRIDE soil suction model and the experimental data.

Figure A3.1 – Comparison between Experimental Data and the STRIDE Soil Suction Model
1.0 INTRODUCTION

Part of the scope of work for STRIDE Phase 4 includes investigation into the pipe/soil interaction of a pipe in a clay soil. The focus of the investigation is soil suction. However, additional work is conducted to gain an insight into vertical stiffness of a clay soil which is detailed in this technical note.

Guidance is given to model vertical pipe/soil interaction (without soil suction) using either a compliant surface or linear/non-linear springs. The vertical pipe/soil interaction is described in two parts, the backbone curve and the force/displacement model. The model is also compared to in-contact cycling experimental data from the CARISIMA II JIP.

2.0 SUMMARY

This technical note presents a method for calculating vertical soil stiffness for pipe/soil interaction in a clay soil. The background theory to develop the stiffness solution is also given, including pipe/soil interaction curve, backbone curve, bearing width, static stiffness and dynamic stiffness. An example of the backbone curve, the pipe/soil interaction curve and associated linear soil stiffness are shown in Figure 2.1.

![Figure 2.1 - Pipe/Soil interaction Curve](image)
The backbone curve shows how the maximum soil resistance force to pipe penetration increases with depth. The pipe/soil interaction curve shows the force and displacement of a pipe moving into the clay with a dynamic motion where the loading direction of the pipe is reversed (see Section 3 for clarification). The dynamic motion is approximated by the linear soil stiffness which is calculated using equation (2.1).

\[ K = k_{stiff} q_U \]  

(2.1)

where

- \( K \)  soil stiffness per meter length (kN/m/m)
- \( k_{stiff} \) stiffness factor, from the experimental data examined \( k_{stiff} = 46 \)
- \( q_U \) bearing capacity at trench depth (kN/m²)

\( k_{stiff} \) may be calculated using equation (5.13) (No Units)

The equation is compared to experimental data from the CARISIMA II JIP. A comparison is made between the stiffness factor \( k_{stiff} \) calculated from an in-contact cycling test and calculated using equation (5.13). A 5 fold difference is found between the experimental and predicted stiffness. As a result it is recommended that further work is required to develop the relationship between the stiffness factor, \( k_{stiff} \) and the mobilisation distance, \( \frac{z_D}{D} \).

### 3.0 PIPE/SOIL INTERACTION

An example of the development of a pipe/soil interaction curve is presented. The right column of Figure 3.1 shows the relationship between the backbone curve, the maximum soil resistance force to pipe penetration at a given depth, and the pipe/soil interaction curve (the force/displacement relationship) of a pipe moving through the soil. The left column shows the vertical motion of the pipe associated with the pipe/soil interaction curve in the right column, as described below:

1. The pipe is suspended above a virgin soil.
2. The pipe penetrates into the soil, plastically deforming it. The pipe/soil interaction curve follows the backbone curve.
3. The pipe moves upwards and the soil responds elastically. The pipe/soil interaction curve breaks away from the backbone curve, the force reduces over a small displacement.
4. The pipe again penetrates the soil, deforming it elastically. The pipe/soil interaction curve follows an elastic loading curve similar to the previous elastic unloading curve of step 3.
5. The pipe again penetrates into the soil, plastically deforming it. The pipe/soil interaction curve rejoins and follows the backbone curve.
Figure 3.1 – Illustration of Pipe/Soil Interaction
4.0 BACKBONE CURVE

4.1 Introduction

A backbone curve shows how the maximum soil resistance force per unit length varies with depth below the seabed surface. Typically backbone curves are constructed using bearing capacity theory.

4.2 Bearing Capacity

A pipe resting on a seabed can be considered as a strip foundation. The soil loading pressure from a strip foundation can be calculated using bearing capacity theory. The formula for the bearing capacity of a strip foundation in an undrained clay soil [1] is given below:

\[ q_U = N_C S_U + \gamma z \]  

\hspace{1cm} (4.1)

where

- \( q_U \) ultimate bearing capacity (kPa)
- \( N_C \) shape and depth factor (-)
- \( S_U \) undrained soil shear strength (kPa)
- \( \gamma \) unit weight of the clay soil (kN/m\(^3\))
- \( z \) depth on the foundation (m)

The \( \gamma z \) term within equation (4.1) is applicable only in foundations that are not backfilled. Skempton's values of \( N_C \) [2], are given in Table 3.1 and may be calculated using equation (4.2). An example of the bearing capacity curve is given in Figure 3.1.

\[ N_C = \text{Min} \left[ 5.14 \times \left( 1 + 0.23 \sqrt{\frac{z}{B}} \right), 7.5 \right] \]  

\hspace{1cm} (4.2)
4.3 Undrained Shear Strength

The shear strength of an undrained clay can be written as a function of depth in the form given below:

\[ S_u = S_{u0} + S_{ug} z \]  

(4.3)

where

- \( S_{u0} \) is the undrained shear strength at surface (kPa)
- \( S_{ug} \) is the undrained shear strength gradient (kPa/m)
4.4 Ultimate Bearing Load

The ultimate bearing load of a pipeline on soft clay, Figure 4.2, can be calculated from a modified form of the bearing capacity equation (4.1) as shown below.

\[ Q_U = q_U \times L \times B \]  \hspace{1cm} (4.4)

where

- \( Q_U \) ultimate bearing load (kN)
- \( L \) length of foundation (m)
- \( B \) bearing width (m)

Bearing load is generally calculated as bearing load per unit length (kN/m) where \( L \) is taken as 1.0m.

**Figure 2.2 – Cross-section of Riser Pipe in a Trench**

There are specific forms of the ultimate bearing load equation for a backfilled trench, equation (4.5), and for a pipe in an open trench, equation (4.6):

\[ Q_U = N_c S_u B \]  \hspace{1cm} (4.5)
\[ Q_U = N_c S_u B + \gamma z B \]  \hspace{1cm} (4.6)

4.5 Bearing Width

For a pipe in a shallow trench \((z < \frac{1}{2} D)\) the bearing width is less than the diameter of the pipe, Figure 4.3.
From Figure 4.3 an equation for half of the bearing width, \( \frac{1}{2}B \), can be developed using Pythagoras’ theorem, equation (4.7) that can be simplified and rearranged in terms of \( z \) and \( D \), equation (4.8)

\[
\frac{1}{2} B = \sqrt{r^2 - (r - z)^2} \tag{4.7}
\]

\[
B = 2\sqrt{Dz - z^2} \tag{4.8}
\]

where

- \( r \) radius of the pipe (m)
- \( D \) outer diameter of the pipe (m)

After the pipe has embedded to a depth greater than \( \frac{1}{2}D \) the bearing width is equal to the pipe diameter.

### 4.6 Backbone Curve

An example of a backbone curve is given in Figure 4.4. A backbone curve can be described using equations (4.5) and (4.6) depending whether the pipe is buried or in an open trench.
5.0 FORCE/DISPLACEMENT RELATIONSHIP

5.1 Soil Stiffness

For SCR analysis two types of soil model are required:

- Static stiffness – the soil stiffness used to model the initial pipe penetration
- Dynamic stiffness – the soil stiffness used to model dynamic pipe/soil interaction

Stiffness may be defined as the ultimate bearing load divided by a mobilisation distance, as shown below:

\[
K = \frac{F}{\Delta}
\]  

(5.1)

where

- \( K \) soil stiffness per unit length (kN/m/m)
- \( F \) force per unit length (kN/m)
- \( \Delta \) displacement (m)

5.2 Linear Stiffness

5.2.1 Static Soil Stiffness Models

Static stiffness is the stiffness required to model the initial penetration of the pipe into a clay soil. It is assumed that the pipe will sink until the bearing load equals the submerged pipe weight, equation (5.2).

\[
m g = Q_s
\]

(5.2)

where

- \( m \) submerged mass per unit length (kg/m)
- \( g \) acceleration due to gravity, 9.81 m/s\(^2\)

From this, the pipe penetration can be calculated (undrained shear strength is a function of soil depth) and a value for the static stiffness obtained, Figure 5.1.
5.2.2 Dynamic Soil Stiffness Models

Dynamic soil stiffness is the stiffness used to model dynamic pipe/soil interaction, Figure 5.2. The dynamic soil stiffness is higher than the static stiffness because the soil has plastically deformed due to the static pipe weight and the pipe has dug itself into the seabed (called trenching).

A simple linear stiffness model can be obtained using the displacement at ultimate capacity, \( z_U \), equation (5.3) [3].

\[
z_U = 0.1B \tag{5.3}
\]

The linear stiffness is obtained using equation (5.1) where \( F = Q_u \) and \( \Delta = z_U \). This simplifies down to the relationships given below:
5.3 Non-linear Stiffness Model

A non-linear stiffness model for pipe/soil interaction may also be used. The model is based on a hyperbolic pipe/soil interaction relationship for a buried pipeline developed by Audibert et al, [2] [4], equation (5.6) and is shown graphically in Figure 5.3.

\[
Q = \frac{z_D}{A' + B' z_D} \quad (5.6)
\]

where

- \( Q \)  non-linear bearing load (kN/m)
- \( z_D \)  dynamic displacement (m)

And

\[
A' = 0.15 \frac{z_U}{Q_U} \quad (5.7)
\]

\[
B' = 0.85 \frac{Q_U}{Q_U} \quad (5.8)
\]
Which when expanded becomes:

\[
Q = \frac{Q_U z_D}{0.15z_U + 0.85z_D}
\]  
(5.9)

Using the above equation (5.7) with equations (5.1) and (5.3) the hyperbolic relationship for bearing load can be simplified into the following equation:

\[
K = \frac{1}{0.015 + 0.85 \frac{z_D}{D}} \times \frac{Q_U}{D}
\]  
(5.10)

The above equation can be further simplified to include a non-linear stiffness factor, \( k_{\text{stiff}} \) for both the backfilled and open trenches:

Pipe in a backfilled trench:

\[
K = k_{\text{stiff}} N_C S_U
\]  
(5.11)

Pipe in an open trench:

\[
K = k_{\text{stiff}} (N_C S_U + \gamma z)
\]  
(5.12)

where the stiffness factor, \( k_{\text{stiff}} \) becomes

\[
k_{\text{stiff}} = \frac{1}{0.015 + 0.85 \frac{z_D}{D}}
\]  
(5.13)

Equation (5.13) is shown graphically in Figure 5.4.
Using the non-linear soil stiffness factor a maximum value for soil stiffness can be determined. As $\frac{z_D}{D} \to 0.0$ equation (5.13) becomes $k_{s\text{iff}} = \frac{1}{0.015} = 66.7$ which is the maximum value for the stiffness factor, $k_{s\text{iff}}$. The non-linear soil stiffness model can also be used to determine a value of $k_{s\text{iff}}$ for the simple linear stiffness model. Using equation (5.3) $\frac{z_D}{D} = 0.1$ which using equation (5.13) calculates a $k_{s\text{iff}}$ value of 10, which is the same as equation (5.5).

6.0 COMPARISON OF THE MODEL WITH EXPERIMENTAL DATA

6.1 Introduction

The CARISIMA (Catenary Riser Soil Interaction Model for global riser Analysis) JIP is developing a vertical and lateral pipe/soil interaction model for riser analysis. Marintek runs the JIP with the pipe pull-out tests conducted by the Norwegian Geotechnical Institute (NGI) in Oslo, Norway.

Within the CARISIMA JIP a number of experiments are conducted to examine the effect of a 0.1016m OD by 0.4064m long pipe being cycled within an open trench. The tests were conducted using the Watchet harbour clay. STRIDE obtained the CARISIMA test data as part of a data exchange [5]. This technical note contains the analysis conducted by 2H for STRIDE on the CARISIMA II in-contact cycling pipe/soil interaction data.
6.2 Experimental Data

A comparison is made between the vertical pipe/soil interaction model and the experimental data from an in-contact cycling test (CARISIMA II Test 2-1). The in-contact cycling test consisted of 10 cycles using force-controlled actuations between -50N and -150N with a period of 10s, Figure 6.1. The parameters for the cyclic test are summarised in Table 6.1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe Diameter, D</td>
<td>0.1016m</td>
</tr>
<tr>
<td>Pipe Length, L</td>
<td>0.4064m</td>
</tr>
<tr>
<td>Trench Depth, z/D</td>
<td>1.0</td>
</tr>
<tr>
<td>Bearing Capacity Factor, Nc from Skempton</td>
<td>6.42</td>
</tr>
<tr>
<td>Undrained Shear Strength, Su</td>
<td>3.86kPa</td>
</tr>
<tr>
<td>Submerged Unit Weight of Soil, γ</td>
<td>5.0kN/m³</td>
</tr>
<tr>
<td>Period of Cycles</td>
<td>10s</td>
</tr>
<tr>
<td>Stress Range</td>
<td>100N</td>
</tr>
<tr>
<td>Displacement Range</td>
<td>0.175mm</td>
</tr>
</tbody>
</table>

Table 6.1 – Parameters from the CARISIMA II Cyclic Loading Test

Figure 6.1 – CARISIMA II In Contact Cyclic Loading
The experimental data is used to back calculate a value of the non-linear stiffness factor, $k_{\text{stiff}}$. Using equation (5.12) for a pipe in an open trench with the data from Table 5.1 a value for $k_{\text{stiff}}$ can be determined:

$$k_{\text{stiff}} = \frac{K}{N_C S_U + \gamma z} = \frac{1407}{6.4 \times 4 + 5 \times 1.0} = 46$$

using equation (4.13) the non-linear soil stiffness model a value for the mobilisation distance can be calculated:

$$\frac{z_D}{D} = \frac{1}{k_{\text{stiff}}} - 0.015 = \frac{1}{46} - 0.015 = 0.008$$

The calculated value for $\frac{z_D}{D}$ is 4.7 times the actual value from the in-contact cycling experiment, Table 6.2. This indicates that there is variability between the test data and equation (5.13). Additional analysis using the available data from both the STRIDE and CARISIMA JIPs is required to further develop the relationship between the stiffness factor, $k_{\text{stiff}}$ and the mobilisation distance $\frac{z_D}{D}$.

<table>
<thead>
<tr>
<th></th>
<th>$z_D$ (mm)</th>
<th>$\frac{z_D}{D}$ (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculated using the pipe/soil interaction model, equation (5.13)</td>
<td>0.8</td>
<td>0.008</td>
</tr>
<tr>
<td>Experimental data</td>
<td>0.175</td>
<td>0.0017</td>
</tr>
</tbody>
</table>

Table 6.2 – Comparison between Mobilisation Distance from Experimental Data and Equation (5.13)

7.0 CONCLUSIONS AND RECOMMENDATIONS

This technical note presents a method for calculating the dynamic soil stiffness for use in SCR analysis. The method is also compared with in-contact cycling experimental data that shows a 5 fold difference between the real data and equation (5.13) – the relationship between the stiffness factor and the mobilisation distance.

A maximum value of the stiffness factor, $k_{\text{stiff}}$ is presented as 67, which if used with equations (5.11) and (5.13) in SCR analysis is assumed to be conservative. From the in-contact cycling experimental data using the 0.1016m OD by 0.4064m long pipe...
with the Watchet Harbour clay a value of 46 is calculated for $k_{\text{stiff}}$. Additional analysis using the available data from both the STRIDE and CARISIMA JIPs is required to properly assess the values of $k_{\text{stiff}}$ to be used in SCR analysis.

8.0 REFERENCES


STEEL CATENARY RISERS - RESULTS AND CONCLUSIONS FROM LARGE SCALE SIMULATIONS OF SEABED INTERACTION

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2H Offshore Engineering Ltd
Woking, Surrey, UK
ABSTRACT

This paper deals with the seabed interaction at the touchdown point (TDP) of deepwater steel catenary risers (SCR's) as investigated within Phase 3 of the STRIDE JIP.

The paper describes back-analysis and conclusions from a test programme that involved a 110m (360-ft) long 0.1683m (6-5/8 inch) diameter SCR at a tidal harbour which had seabed properties similar to those of the deepwater Gulf of Mexico. The top end of the pipe string was actuated using a PLC controller to simulate the wave and vessel drift motions of a spar platform in 1000m (3,300-ft) water depth, both in-line and transverse to the SCR plane. The pipe was fully instrumented to provide tensions and bending moments along its length. Tests were performed at high and low tide.

A pipe/soil interaction model for soil suction was used to predict and back-analyse the response of the harbour test riser. The test data and analytical models achieved good correlation between the tensions and bending moments, indicating that the model could be used to predict suction response from both wave and slow drift vessel motions.

KEY WORDS: Steel Catenary Riser (SCR), Touchdown Point (TDP), Soil Suction

Introduction

Deepwater oil and gas fields usually have seabeds of soft clay. ROV surveys of installed catenary risers have shown deep trenches cut into the seabed beyond the TDP, apparently caused by the dynamic motion of the riser.

Storm and current action on a deepwater production vessel can pull the riser upwards from its trench, or laterally against the trench wall. The suction effect of the soft seabed on the riser, coupled with trench wall interaction, could cause an increase in the local riser stresses (due to tighter riser curvatures and higher tensions) than those predicted ignoring these effects.

As part of the STRIDE III JIP, 2H Offshore Engineering Ltd conducted a test programme to investigate the effects of seabed interaction on catenary riser response and wall stresses. The objective was to assess the importance of seabed/riser interaction, and to produce finite element (FE) analysis techniques to predict the measured response.

Pre-Analysis

Initially, FE analysis was performed to predict the motions of a 6-inch diameter SCR attached to a spar platform in 1000m (3,300-ft) water depth in the Gulf of Mexico, Figure 1. Day-to-day and extreme environmental load-sets were applied using the FE program Flexcom-3D (MCS, 1999), including wave and drift effects both in and out of the riser plane. The riser motions near the seabed were recorded as output from these analyses, and in particular the local velocity of the riser as it peels away from the seabed.

A second FE model was then used to simulate the planned test set-up. This was comprised of a welded steel pipe that represented the bottom 110m (360-ft) of the full-scale riser, Figure 2. The model simulated a linear actuator at the top end. The actuation cycles were varied within the FE model until similar SCR motions were obtained for both the reduced size model and the full depth case. These actuator motions were then used in the design of the actuator rig for the intended tests, allowing the base of a deepwater riser to be simulated at full scale.

Test set up

The test programme was conducted at a harbour location in the west of England. Here the seabed is known to have properties similar to a deepwater Gulf of Mexico seabed. This is made up of soft clay, with an undrained shear strength of 3 to 5 kPa, and a naturally consolidated shear strength gradient below the mudline. Further geotechnical properties are given in Table 1. The sea current velocity in the test area as the harbour filled or emptied was almost negligible, and any trenches formed by the testing remained unchanged over numerous tide cycles.

<table>
<thead>
<tr>
<th>Geotechnical Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content, w</td>
<td>104.7%</td>
</tr>
<tr>
<td>Bulk Density, ρ</td>
<td>1.46 Mg/m³</td>
</tr>
<tr>
<td>Dry Density, ρ_d</td>
<td>0.73 Mg/m³</td>
</tr>
<tr>
<td>Particle Density, ρ_s</td>
<td>2.68 Mg/m³</td>
</tr>
<tr>
<td>Liquid Limit, w_l</td>
<td>87.6%</td>
</tr>
<tr>
<td>Plastic Limit, w_p</td>
<td>38.8%</td>
</tr>
<tr>
<td>Plasticity Index, I_p</td>
<td>48.9%</td>
</tr>
<tr>
<td>Average Organic Content</td>
<td>3.2%</td>
</tr>
<tr>
<td>Specific Gravity, G_s</td>
<td>2.68</td>
</tr>
<tr>
<td>Undisturbed Shear Strength at 1D</td>
<td>3.5 kPa</td>
</tr>
<tr>
<td>Remoulded Shear Strength at 1D</td>
<td>1.7 kPa</td>
</tr>
<tr>
<td>Sensitivity of Clay at 1D</td>
<td>3.3</td>
</tr>
<tr>
<td>Coefficient of Consolidation, c_v at 1D</td>
<td>0.5 m²/year</td>
</tr>
<tr>
<td>Coefficient of Volume Compressibility, m_v at 1D</td>
<td>15 m³/MN</td>
</tr>
</tbody>
</table>

Table 1 – Geotechnical Parameters of Clay Soil

A 110m (360-ft) long 0.1683m (6-5/8 in) diameter welded steel "riser" was suspended from an actuator on the harbour wall, Figures 2 and 3, and run out across the seabed to a set of mud anchors. Further pipe details are given in Table 2. The seabed over this area was flat and undisturbed, and careful probe tests were done to check that there were no hidden obstacles below the mudline.

<table>
<thead>
<tr>
<th>Test Rig Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe outer diameter</td>
<td>0.1683m (6-5/8&quot;)</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>6.9mm</td>
</tr>
<tr>
<td>Pipe material</td>
<td>APL 5L Grade B, 448.2x10^6 N/m² yield</td>
</tr>
<tr>
<td>Height of nominal position above seabed</td>
<td>9.65m</td>
</tr>
<tr>
<td>Length of chain at actuator</td>
<td>3.35m</td>
</tr>
<tr>
<td>Length of pipe</td>
<td>1.95m</td>
</tr>
<tr>
<td>Mean water level</td>
<td>3.5m</td>
</tr>
</tbody>
</table>

Table 2 – Summary of Test Rig Parameters

The test set-up allowed the use of a number of virgin test corridors at the flattest part of the harbour seabed. It was important that these corridors were undisturbed before the testing. To ensure this, the riser was floated to the various positions using temporary buoyancy, then the outgoing tide allowed it to settle onto the seabed.

The riser was fixed at its top end to an actuator unit. This comprised a heavy-duty truss frame with a 3m (10-ft) linear ball
screw driven from one end by a motor with displacement feedback control via PLC, Figure 4. The riser was attached to the ball screw nut via an adjustable cable. This allowed the top tension in the riser to be tuned to the prescribed value of 56.5kN, which set the TDP at 64.2m from the actuator. The linear screw could be swivelled to operate in vertical or horizontal directions. This system applied the prescribed motions accurately to the top end of the harbour test riser, and produced the vertical and lateral pipe motions which were necessary at the seabed. This meant linear ramps, simulating vessel drift, and sinusoidal motions of different amplitudes and frequencies, simulating wave loading. In addition the whole actuator frame was designed to move on a set of 10m long rails, simulating a large transverse excursion of the vessel and pulling the riser laterally from its trench while pipe stresses were monitored.

The primary instrumentations comprised full bridge strain gauge sets which were welded at 13 axial positions along the riser and spanned the dynamic TDP area, Figure 2. Each position provided vertical and horizontal bending strain on the pipe. In addition, a triaxial accelerometer unit was mounted just above the nominal TDP. There were tension load cells at the top and bottom of the pipe string, and strain gauges measuring shear force at the connection between pipe and actuator. All instrumentation was hardwired to a multi-channel logging station which was able to monitor in real-time at 40 Hz.

Test Program

The test corridors used included: an open trench, an artificially deepened trench, a backfilled trench and a rigid seabed, Table 3. For each test corridor a series of tests was conducted to examine the effects of slow drift (pull up and lay down tests) and dynamic motions (day-to-day and second order motions). Table 4 has a definition of the actuations referred to within this paper.

<table>
<thead>
<tr>
<th>Actuation Reference</th>
<th>Offshore Equivalent Motion</th>
<th>Travel at Actuator</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic @ near/nominal/far</td>
<td>Heaving storm wave about either the 0.5% WD near, nominal, 1.1% far vessel position</td>
<td>Vertical sine wave, +/- 0.4m, 25 second period about the -0.4m datum, 0m datum, +1.0m datum</td>
</tr>
<tr>
<td>Lateral dynamic</td>
<td>Surging or swaying storm wave about nominal</td>
<td>Horizontal sine wave, 0m datum, +/- 0.4m, 18 second period</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Corridor</th>
<th>Test Corridor Title</th>
<th>Description/Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>First Trench</td>
<td>Initial trials, no data recorded</td>
</tr>
<tr>
<td>2</td>
<td>Open Trench</td>
<td>Formed naturally by riser self weight and vertical/lateral motions</td>
</tr>
<tr>
<td>3</td>
<td>Artificially Deep Trench</td>
<td>The trench was artificially deepened</td>
</tr>
<tr>
<td>4</td>
<td>Backfilled Trench</td>
<td>The artificially deep trench was backfilled with clay</td>
</tr>
<tr>
<td>5</td>
<td>Rigid Seabed</td>
<td>Steel planks were placed over the trench and under the riser to simulate a rigid seabed</td>
</tr>
</tbody>
</table>

Table 3 – Description of Trench Corridors

The test matrix for test corridors 2 – 5 is shown in Table 5. The matrix shows that pull up and lay down tests were conducted on every test corridor, however the wave motions were only conducted on test corridor 2 (open trench) and test corridor 5 (rigid seabed).

The pull up and associated lay down tests were typically conducted as a series of 5 consecutive pairs. The first pull up test is considered to be on undisturbed clay as the riser was allowed to consolidate the clay soil in the trench. Table 6 shows the consolidation time and the sea level of the first pull up tests. The subsequent tests in the pull up and lay down series are considered to be on remoulded clay.

<table>
<thead>
<tr>
<th>Test Corridor</th>
<th>Description</th>
<th>Open Trench</th>
<th>Artificial Trench</th>
<th>Backfilled Trench</th>
<th>Rigid Seabed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pull-up / Lay down</td>
<td>3, 4, 7, 8, 10, 11, 13, 14 (C,D)</td>
<td>3, 4, 5, 6</td>
<td>1, 2</td>
<td>1, 2</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Corridor</th>
<th>Description</th>
<th>Open Trench</th>
<th>Artificial Trench</th>
<th>Backfilled Trench</th>
<th>Rigid Seabed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic @ Near</td>
<td>5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td>Dynamic @ Nominal</td>
<td>6</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>4</td>
</tr>
<tr>
<td>Dynamic @ Far</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>5</td>
</tr>
<tr>
<td>Lateral Pull Out</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Lateral Dynamic</td>
<td>16</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

Table 4 – Actuation Definitions and Parameters with Equivalent SCR motions

<table>
<thead>
<tr>
<th>Test Corridor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pull-up / Lay down</td>
<td>1, 2, 13, 14 (A,B)</td>
</tr>
<tr>
<td>Dynamic @ Near</td>
<td>9, 12</td>
</tr>
<tr>
<td>Dynamic @ Nominal</td>
<td>-</td>
</tr>
<tr>
<td>Dynamic @ Far</td>
<td>-</td>
</tr>
<tr>
<td>Lateral Pull Out</td>
<td>17, 18, 19, 20</td>
</tr>
<tr>
<td>Lateral Dynamic</td>
<td>15</td>
</tr>
</tbody>
</table>

Table 5 – Test Matrix with Test Reference Numbers
Results

The results from the harbour test riser are presented as bending moment traces versus actuator position at strain gauge locations. Figure 5 shows an example of the bending moment data from a strain gauge during a first pull up test and the associated lay down test. A negative bending moment corresponds to a sagging bend in the riser. If the lay down test is considered to represent the ‘no soil suction’ case and the pull up test representing the ‘with soil suction’ case the two bending moment traces can be compared directly. Both the pull up and lay down bending moment traces start from the –0.8m actuator position with bending moments of around -0.5 kNm. The lay down bending moment trace decreases steadily to a minimum value of -5.5kNm at an actuator position of 0.5m where it levels off. In contrast, the pull up bending moment trace does not change until the actuator has moved to the –0.3m actuator position. This indicates that the soil suction force is holding the riser in place. The bending moment then decreases rapidly to peak at -11 kNm at an actuator position of 0.5m, which is twice the lay down bending moment. The pull up bending moment trace then increases to join the lay down bending moment trace at the 1.2m actuator position.

From this example, it can be seen that the peak bending moment during a near to far pull up test is twice that of the peak bending moment seen during the associated lay down test.

Figure 6 shows the bending moment trace of a pull up and lay down test pair on the rigid seabed. It can be seen that the pull up and lay down tests are virtually identical, and shows that the peak in the bending moment trace during the pull up test with the riser on the clay soil is due to soil suction, and not a result of the actuation system or hysteresis/inertia effects.

The effect of soil suction on a first pull up and the associated lay down test are shown in Figure 8. The location of strain gauge positions A, D, F, J, K and M along the riser are shown in Figure 7. The pull up test (2-10) and the lay down test (2-11) were conducted in test corridor 2, with an actuator pull up rate of 0.1m/s after 72 hours consolidation. This simulated a slow drift motion. Descriptions of the pull up and lay down bending moments follow:

**Position A** – this location is free hanging when the riser is in the near (lowest) actuation position. As the riser is pulled up the strain gauge shows a small decrease (around 0.3kNm) in the bending moment as it is pulled up into a straighter part of the catenary.

**Positions D and F** – these locations show the greatest change in bending moment due to soil suction. They were positioned close to the nominal TDP, are in contact with the seabed in the near riser position and are free hanging when the riser was pulled up.

**Positions J and K** – these locations are in contact with the seabed for much of a pull up test, only becoming free hanging when the actuator position is close to the 1.0m. However they do show that the soil suction holds the riser to the seabed.

**Position M** – This location is in contact with the soil at both near and far actuator positions.

The influence of repeated loading, pull up velocity and consolidation time on soil suction was also investigated. The observations on these aspects of response are given below:

**Repeated Loading** – Figure 9 shows the bending moment response of strain gauge location D during a first pull up (test 3-5), a sixth pull up (test 3-5E) and an associated lay down (test 3-6). These shown that after the first pull up soil suction increases the magnitude of the bending moment peak by 85%. However, for the sixth pull up the peak bending moment increase drops to 20%. This shows a 76% reduction in the bending moment response, and indicates that the soil suction force has reduced between the first and sixth pull up tests.

Figure 10 shows a summary of the minimum bending moments from pull up test series 3-5 compared to lay down test 3-6. It is shown that the soil suction force reduces by 66% between the first and second pull up tests, and then reduced further by around 4% for each subsequent test.

**Pull Up Velocity** – Consecutive pull up tests 2-1C (fourth pull up) and 2-1D (fifth pull up) were conducted after repeated loading with pull up velocities of 0.1m/s and 0.01m/s, respectively. The results, Figure 11, show that on remoulded clay the pull up velocity has little effect on the bending moment response.

**Consolidation Time** – Figure 12 shows the effect of consolidation time on strain gauge positions C and D during pull up tests 3-3 (4 hours consolidation) and 3-5 (12 hours consolidation). With increased consolidation time the magnitude of the bending moment response at strain gauge location C increases by 3kNm (58%) and at location D by 2kNm (23%).

From study of the harbour test data additional interaction effects was observed due to soil suction, including suction release and a suction kick, both of which are described below:

**Suction Release** – After the pull up test actuation was complete (the pipe had been pulled to the top of the actuator) the bending moment response at strain gauges J and K was seen to continue to change. This effect, not seen on the lay down tests, is due to the mobilised suction force dissipating and allowing the riser to move into the static equilibrium position.

Figure 13 shows how the bending moment response of strain gauge locations C, J and K and the corresponding actuator position change with time. The vertical blue lines show the start and end of the pull up test. It can be seen that the bending moments do not change over the 10s before the pull up tests starts. Once the test begins all strain gauge locations show a bending moment response similar to those previously observed, Figure 8. After the tests has finished the bending moment response at strain gauge C remains constant. However the bending moment response of strain gauges J and K continue to change for 15s and 18s respectively.

This indicates that it a riser is left statically after soil suction has been mobilised the suction slowly dissipates and the riser moves into the equilibrium state, which has little or no soil suction.

**Suction Kick** – Figure 14 shows the bending moment response of fast pull test 3-5, conducted with a sea level of 2.6m. It can be seen that when the actuator moves past 0.6m the bending moment

<table>
<thead>
<tr>
<th>Consolidation Time (hours)</th>
<th>Sea Level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.0 – 1.5</td>
</tr>
<tr>
<td>4</td>
<td>1.5 – 2.0</td>
</tr>
<tr>
<td>12</td>
<td>2.0 – 2.5</td>
</tr>
<tr>
<td>16</td>
<td>2.5 +</td>
</tr>
<tr>
<td>72</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 6 – Summary of First Pull Up Tests
responses of strain gauges A, C, D and J start to oscillate. This appears to be due to a rapid release of soil suction, and is termed a suction kick.

These observations of the test data shows some of the effects that influence the soil suction force, including repeated loading, changing the pull up velocity and the length of the consolidation time. The tests also showed some effects of soil suction on the harbour test riser, including the suction kick and suction release. The test data was also used to refine and calibrate the 2H Offshore Engineering Ltd soil suction model.

**Analytical Modelling**

An analytical model of the harbour test riser was produced to calibrate the 2H Offshore Engineering Ltd soil suction model. The analysis was conducted using both the ABAQUS and ANSYS finite element codes.

The soil suction curve used in the analytical modelling was the upper bound curve based on the previous STRIDE 2D pipe/soil interaction work, Willis (2001). The soil curve, Figure 15, consists of 3 sections: suction mobilisation, the suction plateau and suction release, described below:

- **Suction mobilisation** – As the riser initially moves upwards the suction force increases from zero to the maximum value
- **Suction plateau** – The suction force remains constant as the riser moves further upwards
- **Suction release** – Under further upward movement the suction force reduces from its maximum to zero at the break-out displacement

The analytical model was created to match the final harbour test riser as closely as possible. The model dimensions were taken from surveys of the riser and the seabed profile conducted during the testing program. The model for each analytical test corridor was then calibrated to the 'as built' riser by changing the length of the top cable.

Comparisons between results obtained using the analytical model and the harbour test riser are conducted using two methods. The first compares the test data from a single strain gauge location to that of a similar point on the analytical model.

The magnitudes of the bending moments at the start of the analytical model are matched to those of the harbour test riser to account for the effects of the uneven seabed. The analysis using no soil suction (green line), Figure 16, is compared to the lay down test (blue line). The analysis using the upper bound soil curve (black line) is compared to the pull up test (red line).

Comparisons between the results from the analytical modelling and test corridors 2 and 4 are shown in Figures 16 and 17 respectively. It can be seen that the analytical model using the upper bound soil suction curve predicts the test data very well.

The second method of comparing test and model results is achieved using the bending moment envelopes from the analytical predictions for the no suction and with suction models. These are compared to the maximum and minimum of the strain gauge locations during pull up and lay down tests, Figure 18. As before, the green and black lines represent the no soil suction and the upper bound soil suction models respectively (Note that the top most black line is on top of the green line). The red and blue lines show the minimum and maximum bending moments from pull up (with soil suction) and lay down (no soil suction) tests. The difference between the red and blue lines is the effect on the bending moment of the soil suction.

It can be seen that the analytical bending moment envelopes compare well to the test data ranges; the no soil suction model predicting the lay down test data ranges well. The effect of soil suction is also predicted well, with strain gauges C, D and F showing a similar response to the upper bound soil model. The analytical model also predict the response of strain gauges J and K, which exhibit a lower change in bending moment during the pull up test than lay down test.

The analysis shows that the effects of soil suction can be predicted using the 2H Offshore Engineering Ltd soil suction model with the appropriate soil suction curve.

**Conclusions**

The full-scale tests provide a valuable basis for evaluation of SCR soil interaction and validation of numerical models. A numerical model has been developed based on small-scale tests and validated using the full-scale response measurements. This suggests that numerical models can be developed to predict full-scale response in various geographical locations based upon small-scale pipe/soil interaction tests using soil representative of the location.

Soil suction is shown to occur and produces differences in bending moment response between a pull up test and a subsequent lay down test. Comparisons of pull up and lay down response from the range of tests conducted produced the following key findings:

- A sudden vertical displacement of a catenary riser at its touchdown point (TDP) after a period at rest could cause a peak in the bending stress that travels along the riser. Such an event may occur from a vessel failed mooring line, or a move away for drill rig access.
- Soil suction forces are subject to hysteresis effects. For example, once the seabed/riser interface has been disturbed, subsequent seabed/riser interface contacts produce less suction effect.
- The soil suction force is dependant on the consolidation time.
- Pull up velocity has little effect on the bending moment response on a remoulded seabed.
- Soil suction can cause effects such as the suction kick
- Following any actions resulting in pull-up, the mobilised soil suction will dissipate, and the riser will move into an equilibrium position with no or little no soil suction.

The analytical modelling of the harbour test riser used a soil suction curve developed from 2D pipe/soil interaction tests with the 2H Offshore Engineering Ltd soil suction model. The analytical models predict the harbour test riser bending moment measurements well.

**ACKNOWLEDGEMENT**

The authors are grateful to the STRIDE JIP Participants for their permission and support in submitting this paper. The views presented in this paper are those of the authors, and may not necessarily represent those of the participants.
STRIDE Phase III details

Lead Engineering Contractor: 2H Offshore Engineering

Participants:

BP Statoil Aker
Chevron Texaco Brown & Root
Conoco TotalFinaElf Single Buoy Moorings
Norsk Hydro Vastar Sofec

Programme manager: Offshore Technology Management

REFERENCES


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Soil Suction Release
In Plane Bending Moment Vs Time

Start of Actuation
(After Slow Start)

End of Actuation

Bending Moment Relaxing after Actuation has Ended

Bending Moment Relaxing after Actuation has Ended

Figure 14 – Evidence of Suction Release with Time

Soil Suction Kick
In Plane Bending Moment Vs Actuator Motion

Bending Moments prior to the Suction Kick
Bending Moments showing evidence of a Suction Kick

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Figure 17 – Comparison of Test Data with Analytical Model for Test Area 4

Figure 18 – Comparison of Test Data and Analytical Bending Moment Envelope
Full scale model tests of a steel catenary riser

C. Bridge¹, H. Howells¹, N. Toy², G. Parke², R. Woods²
¹ 2H Offshore Engineering Ltd, Woking, Surrey, UK
² School of Engineering, University of Surrey, Guildford, GU2 7XH, UK

Abstract

Steel catenary risers (SCRs) are an enabling technology for deepwater oil and gas production. Tools to analyse and design SCRs are available which show that the point where the riser first touches the soil, termed the touchdown point (TDP) is critical. However our understanding of fluid/riser/soil interaction is limited, hence the oil and gas industry has concerns regarding the levels of conservatism in SCR design, and margins of safety. The purpose of this study is to examine the interaction between a pipe (representing a section of the SCR), a clay seabed, and the surrounding seawater.

This paper documents some of the results and observations from the full scale harbour test riser experiments which examined the 3D effect of fluid/riser/soil interaction around the TDP. The riser, a 110m (360-ft) long 0.1683m (6-5/8 inch) diameter pipe, was draped from an actuator on the harbour wall to an anchor point on the seabed. The top end of the pipe string was actuated using a programmable logic controller (PLC) to simulate the wave and vessel drift motions of a spar platform in 1000m (3,300-ft) water depth, both in-line and transverse to the SCR plane. The pipe was fully instrumented to provide tensions and bending moments along its length.

Observations from the harbour tests show that a trench forms around the TDP. Evidence collected shows that the trench created was tear-drop shaped, with a maximum width of 2.5 riser diameters and a maximum depth of 1.2 diameters. The trench was thought to be created from a combination of the applied vessel motions and fluid flow across the riser and the seabed, however the exact trenching mechanisms are unknown.

The work was conducted as part of the successful STRIDE JIP (Steel Risers in Deepwater Environments Joint Industry Project).
1 Introduction

1.1 Steel catenary risers

A SCR is a long steel pipe that hangs freely between the seabed and a floating production system. The top of a SCR is connected to the floating production system, where it hangs at a prescribed top angle. The riser is free-hanging and gently curves down to the seabed to the TDP. At the TDP the SCR buries itself in a trench and then gradually rises to the surface where it rests, and is effectively a static pipeline. SCRs may be described as consisting of three sections as shown in Figure 1, below:

- Catenary zone, where the riser hangs in a catenary section
- Buried zone, where the riser is within a trench
- Surface zone, where the riser rests on the seabed

![Figure 1: General Catenary Arrangement](image)

Predicting the shape and general forces on a SCR is a relatively simple process, the most basic of which is to solve standard catenary equations. More detailed analysis of risers can be conducted using non-linear finite element analysis programs. Most specialist state-of-the-art riser analysis codes use either rigid or linear elastic contact surfaces to simulate the seabed, which model vertical soil resistance to pipe penetration, horizontal friction resistance and axial friction resistance. A rigid surface generally gives a conservative result since it is unyielding, while the linear elastic surface is a better approximation of a seabed.
1.2 Vessel motions

The vessel from which the SCR hangs is generally a floating production vessel, and as such is subject to wave, current and wind loading. During normal operating conditions the SCR connects to the vessel via either a flex joint or a taper stress joint. These transfer the dynamic motions of the vessel directly to the top of the SCR, which causes the TDP to move along the riser. It has been found that of all the vessel motions, heave causes the greatest stress fluctuations at the TDP [1]. Analysis has shown that a dynamic heave motion of ±1m amplitude can cause the TDP on a SCR in 1000m water depth to move horizontally by 10m. The main forms of loading on vessels are described below:

- First order motions – wave frequency motions caused by wave action on the vessel.
- Second order motions – low frequency motions caused by wind gusts, often referred to as slow drift motions.
- Static offset – displacement resulting from mean environmental loads such as currents, waves and winds, or system failures, such as failed mooring lines.

In addition to the vessel loads the current acts directly on the SCR. This causes the riser to flex in the direction of the current, and can invoke high frequency vortex induced vibration (VIV) motions in the riser.

1.3 Touchdown point

Deepwater oil and gas fields usually have seabeds of soft clay. ROV surveys of installed SCRs have shown deep trenches cut into the seabed beyond the TDP. The mechanisms that create these trenches are unknown, however they are thought to be produced by the dynamic motions of the riser combined with the scouring and sediment transportation effects of the seabed currents.

Storm and current action on a deepwater production vessel can pull the riser upwards from its trench, or laterally against the trench wall. This interaction could cause an increase in the local riser stresses (due to tighter riser curvatures and higher tensions) than those predicted ignoring the seabed trench.

1.4 Harbour tests

As part of the STRIDE III JIP, 2H Offshore Engineering Ltd conducted a full-scale test programme to investigate the effects of fluid/riser/soil interaction on catenary riser response and wall stresses, Figure 2 at the TDP. The objective was to assess the importance of fluid/riser/soil interaction, and to produce finite element (FE) analysis techniques to predict the measured response.
2 Harbour test riser

The test programme was conducted over 3 months at a harbour location in the west of England. A 110m (360-ft) long 0.1683m (6-5/8 in) diameter, 6.9mm wall thickness welded steel (APL 5L Grade B) riser was suspended from an actuator on the harbour wall and run out across the seabed to a set of mud anchors, Figure 3. The seabed over this area was flat and undisturbed, and careful probe tests were done to check that there were no hidden obstacles below the mudline.

![Figure 3: Harbour test set up and the locations of strain gauges A to M](image)

The harbour tests riser was completely instrumented with 13 sets of strain gauges measuring vertical and horizontal bending strain which spanned the TDP area, Figure 3, and load cells measuring the tensions and shear forces at the
actuator and the tension at the anchor. In addition triaxial accelerometers were placed on the actuator and at strain gauge position A. All instrumentation was hard wired back to a real time, 40hz multi-channel logging station.

2.1 Marine parameters

The mean sea level was 3.5m above the anchor. The current velocity due to the tides in the test area as the harbour filled or emptied was low. Tests were conducted at both high and low tides.

2.2 Geotechnical parameters

The Watchet Harbour seabed is known to have properties similar to a deepwater Gulf of Mexico seabed. This is made up of soft clay, with an undrained shear strength of 3 to 5 kPa, a sensitivity of 3, a plasticity index of 39%, and a naturally consolidated shear strength gradient below the mudline. Further geotechnical properties are given by Bridge & Willis [2].

2.3 Test program

The harbour tests were conducted over a 6 week period on numerous test corridors including an open trench, an artificially deepened trench, a backfilled trench and on a rigid seabed. For each test corridor a series of tests was conducted to examine the effects of slow drift (pull up and lay down tests) and dynamic motions (day-to-day and second order motions), Table 1.

Table 1: Actuation definitions and parameters with equivalent SCR motions

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<tr>
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<td>Heaving storm wave about either the 0.5% WD near, nominal, 1.1% far vessel position</td>
<td>Vertical sine wave, +/- 0.4m, 25 second period about the -0.4m datum, 0m datum, +1.0m datum</td>
</tr>
<tr>
<td>Pull-up</td>
<td>Spar failed mooring drift speed, near 0.8% to far 1.4% WD</td>
<td>-0.8m to +1.4m @ 0.1m/s and 0.01m/s</td>
</tr>
<tr>
<td>Lay-down</td>
<td>Spar failed mooring drift speed, far 1.4% to near 0.8% WD</td>
<td>+1.4m to -0.8m @ 0.1m/s and 0.01m/s</td>
</tr>
</tbody>
</table>

3 Typical results from the harbour test riser

The results from the harbour test riser are presented as bending moment traces versus actuator position at strain gauge locations. Comparisons are made between the bending moment data from a strain gauge during pull up and lay
down tests. A negative bending moment corresponds to a sagging bend in the riser.

An example of a typical bending moment trace with actuator position is given in Figure 4. It shows that when the actuator is at -0.8m, at the bottom of the vertical stroke, the bending moment is around 0.5kNm. As the actuator moves upwards the bending moment is constant until the actuator reaches -0.6m, after which the bending moment reduces to a peak of -6kNm at an actuator position of 0.8m. The bending moment then reduces to -5.5kNm at an actuator position of 1.4m.

Initially the pipe location is on the seabed, in the surface zone. Then as the actuator moves the top of the pipe upwards the pipe location moves into and through the buried zone, until at the end of the actuation the pipe location is free hanging in the catenary zone. During this actuation the TDP is observed to move 25m towards the anchor. Further results from the harbour tests have been presented by Bridge & Willis [2].

![Bending Moment Trace at Strain Gauge D](image)

Figure 4: Comparison of pull up and lay down tests on a rigid seabed

4 Observations of riser trenches

When the harbour test riser was initially placed on the seabed the soil deformed to create a close fitting trench around the pipeline. This close fitting trench was observed at low tide after the riser had been floated into place.

During the testing the trench was observed to deepen and widen around the TDP. A photograph of the trench formed is given in Figure 5. This shows the section of the harbour tests riser as it passes from the catenary zone, through the TDP into the buried zone and then into the surface zone where the pipe is connected to the anchor. The trench formed starts where the riser first touches
the soil when the actuator is at its lowest position (which it was between most
tests). The trench extends towards the anchor and the width increases from 1
diameter to a maximum of 2.5 diameters over a distance of 20m. The trench then
reduces in width to 1 diameter over the next 40m at which point it is considered
to be a static pipeline in the surface zone.

Figure 5: The harbour test riser in a naturally occurring seabed trench at low tide

Two close ups of the trench are shown in Figure 6. Both photographs are
taken from the widest part of the trench, Photograph A faces the anchor and the
surface zone while Picture B faces the actuator and the catenary zone. The
photographs show that there is no build up of soil around the top of the trench,
which may be expected if the riser had been pushed into the trench walls by the
tidal currents. It can also be seen that the tops of the trench wall are curved
which could have been worn away by the tidal currents.

Measurements taken during the testing program, Figures 7 and 8, show the
trench to be tear-drop shaped and that the maximum depth and width increases
over the 6 week testing period from 0.5 diameters to 1.2 diameters and from 1
diameter to 2.5 diameters respectively.

The mechanisms that created the trench are unknown, however there are
many possibilities including:

- The dynamic motions applied by the actuator, representing the vessel
  motions, may have dug the trench. In addition any vertical motion at the
TDP would cause the water beneath the riser to be pumped out of the trench, carrying sediment with it.

- The flow of the tides may have scoured and washed away the sediment around the riser.
- The flow of the seawater across the riser can cause VIV (which was observed when the tide came in or went out). This high frequency motion could act like a saw, slowly cutting into the seabed.
- When the harbour test riser is submerged the buoyancy force causes the riser to lift away from the seabed. Any lose sediment in the trench or attached to the riser would be washed away.

Figure 6: Close up photographs of the trench at low tide
Figure 7: Measurements of trench depth

Figure 8: Measurements of trench width
5 Conclusions

The full-scale tests provide a valuable basis for evaluation of SCR fluid/riser/soil interaction and validation of analytical models. A comparison of the pull up and lay down tests on the rigid seabed shows that the bending moment data is consistent between similar tests.

Evidence collected from the harbour tests show that over a period of 6 weeks a trench was created near the TDP which was tear-drop shaped, with a maximum width of 2.5 diameters and a maximum depth of 1.2 diameters. The trench is thought to be created from a combination of the applied motions and fluid flow across the riser and the seabed, however the exact trenching mechanisms are unknown. Further work is required to determine the primary trenching mechanisms so that accurate predictions can be made to reduce the conservatism in SCR design.

6 Acknowledgements

The authors would like to thank 2H Offshore Engineering ltd for the opportunity to write and present this paper. The views presented in this paper are those of the authors, and may not necessarily represent those of the STRIDE JIP.

7 References


Steel Catenary Riser Touchdown Point Vertical Interaction Models
Christopher Bridge, 2H Offshore Engineering Ltd
Katherine Laver, Ed Clukey, Trevor Evans, BP

Abstract
Steel catenary risers (SCR) are an enabling technology for deepwater environments. Tools to analyse and design SCRs are available which show that the point where the riser first touches the soil, termed the touchdown point (TDP), exhibits complex behaviour that has been the subject of a number of recent research programmes. The soil parameters used in SCR analysis can have a significant effect on riser response, especially the predicted fatigue life.

If soil parameters and analytical models are chosen too conservatively they can make the predicted fatigue life unrealistically low, conversely using non-conservative soil parameters and soft soil models results in fatigue lives that may be unrealistically high.

This paper describes state of the art vertical pipe/soil interaction models developed for use in SCR analysis. These model pipe movement vertically downwards (soil stiffness) and vertically upwards (soil suction). The models are based upon test data from the STRIDE and CARISIMA JIP's and information from existing papers. The models are currently being used in many Gulf of Mexico deepwater projects that involve SCRs.

Introduction
The seabed models used in SCR analysis can have a large effect on the predicted riser fatigue life. Case studies on generic SCRs conducted within the STRIDE JIP [1] show that the predicted fatigue damage is dependant on the value of soil stiffness used, Figure 1. High values of soil stiffness (around 10,000kN/m/m) produce fatigue damages similar to those calculated using a rigid seabed. If the soil stiffness is reduced to 1,000kN/m/m the fatigue damage reduces by around 30%, an increase in fatigue life of 43%. If the soil stiffness is further reduced to 100kN/m/m the fatigue damage is around 45% of the rigid seabed, and increase in the predicted fatigue life of 120%.

This shows that if the level of soil stiffness used in SCR analysis is too high then the predicted fatigue life may be too low, conversely if the soil stiffness is low then the analysis may not be conservative, or representative.

The STRIDE case studies also investigated the effect of soil suction, the soil resistance force to the pipe moving vertically upwards on SCR fatigue life. The studies show that soil suction has a small effect on fatigue damage, but a large effect on extreme stress when the riser is pulled away from the seabed. Events that cause this type of motion include slow drift caused by a failed mooring line or pulling the pipe away from the seabed during installation operations.

The soil stiffness and soil suction models presented in this paper are developed using STRIDE and CARISIMA test data in combination with information taken from published literature. The work was conducted during the STRIDE JIP, with additional work sponsored by BP.

Pipe / Soil Interaction
An example of the development of a pipe/soil interaction curve with an unloading/reloading cycle is presented in Figure 2. The right hand column of Figure 2 shows the relationship between the backbone curve (the maximum soil resistance force to pipe penetration at a given depth) and the pipe/soil interaction curve (the force/displacement relationship) of a pipe moving through the soil. The left hand column shows the vertical motion of the pipe associated with the pipe/soil interaction curve in the right column, in steps as described below:

1. The pipe is suspended above a virgin soil
2. The pipe penetrates into the soil, plastically deforming it. The pipe/soil interaction curve follows the backbone curve.
3. The pipe moves upwards and the soil responds elastically. The pipe/soil interaction curve breaks away from the backbone curve, the force reduces over a small displacement.
4. The pipe again penetrates the soil, deforming it elastically. The pipe/soil interaction curve follows an elastic loading curve similar to the previous elastic unloading curve of step 3.
5. The pipe again penetrates into the soil, plastically deforming it. The pipe/soil interaction curve rejoins and follows the backbone curve.
The previous example showed the force/displacement curve developed during an in-contact cycle, where the pipe does not lose contact with the soil. An example showing the force/displacement curves developed during pipe penetration and a cycle with break-out, where the pipe loses contact with the soil is given in Figure 3 and described below.

1. **Penetration** – the pipe penetrates into the soil to a depth where the soil force equals the penetration force. The penetration force displacement curve follows the backbone curve. The soil deforms plastically.

2. **Unloading** – the penetration force reduces to 0 N allowing the soil to swell as the pipe moves upwards.

3. **Soil suction** – as the pipe continues to move upwards the adhesion between the soil and the pipe causes a tensile force that resists the pipe’s motion. The adhesion force quickly increases to a maximum then reduces to 0 N as the pipe moves vertically upwards and out of the trench.

4. **Re-penetration** – the pipe penetrates into the existing trench that was created during the initial penetration. The re-penetration force/displacement curve has zero force when the pipe re-enters the trench, only increasing the interaction force when the pipe re-contacts the soil. The pipe/soil interaction force then increases until it rejoins the backbone curve at a lower depth than the previous penetration. Any further penetration follows the backbone curve.

**Backbone Curve**

A backbone curve shows how the maximum compressive soil resistance force per unit length varies with depth below the seabed surface as a pipe is continuously pushed into the soil for the first time. Typically backbone curves are constructed using bearing capacity theory of strip foundations [2]. The equations for calculating the backbone curve in undrained clay soils are given below.

\[
Q_U = q_U B
\]

\[
q_U = N_C S_U + \gamma z
\]

where

- \( Q_U \) ultimate bearing load per unit length of pipe
- \( q_U \) ultimate bearing pressure
- \( B \) bearing width of pipe
- \( N_C \) non-dimensional shape and depth factor
- \( S_U \) undrained shear strength of soil
- \( \gamma \) submerged unit weight of the soil
- \( z \) depth of pipe invert

The \( \gamma z \) term in the bearing capacity equation is applicable in pipelines that are not backfilled or buried naturally. Values for \( N_C \) can be calculated using Skempton’s method using the following formula [3].

\[
N_C = \text{Min} \left[ 5.14 \times \left( 1 + 0.23 \frac{z}{B} \right), 7.5 \right]
\]

The bearing width of a pipe is typically equal to the external diameter of the pipe after the pipe has penetrated to a depth greater than half of the external diameter. If the penetration depth is less than \( \frac{1}{2} D \) the bearing width is calculated using the following formula.

\[
B = 2\sqrt{Dz - z^2}
\]

where

- \( D \) external diameter of riser

The undrained shear strength of a soil can be written as a function of depth below seafloor in the form given below.

\[
S_U = S_{U0} + S_{UG} z
\]

where

- \( S_{U0} \) undrained shear strength at surface
- \( S_{UG} \) undrained shear strength gradient

For most practical purposes the undrained shear strength used in Equation (2) may be taken as the strength at pipe invert level.

**Soil Stiffness**

Most specialist state-of-the-art riser analysis codes use either rigid or linear elastic contact surfaces to simulate the seabed. SCR analysis is generally conducted using a linear elastic surface since it is a better approximation of the seabed than a rigid surface. To use the linear elastic surface as the seabed a linear representation of the non-linear pipe/soil interaction curves is required. This can be achieved by determining appropriate values of soil stiffness, which can be defined as the ultimate bearing load divided by a distance, as shown below:

\[
K = \frac{F}{\Delta}
\]

where

- \( K \) soil stiffness per unit length
- \( F \) force per unit length
- \( \Delta \) displacement

There are three types of soil stiffness used when modelling pipe/soil interaction: static, large displacement dynamic and small displacement dynamic. Examples of linear static and dynamic soil stiffness are given in Figure 4 and described below.

Static soil stiffness, or secant stiffness, is the stiffness required to model the initial pipe penetration into a virgin seabed.

Large displacement dynamic soil stiffness is the stiffness required to model cyclic TDP motions where breakout occurs. It needs to account for the initial plastic deformation of the soil and is typically a modified secant stiffness.

Small displacement dynamic soil stiffness, which can be either the tangent stiffness for very small displacements or...
secant stiffness for larger displacements, is used to model any in-contact cyclic pipe/soil interaction after penetration or re-penetration have taken place.

**Static Stiffness**

Static soil stiffness is used to estimate the initial penetration of an SCR in a virgin seabed. Typically the submerged weight of the pipe per unit length is equated to the backbone curve, which is solved for depth. For the embedment at the TDP on a rigid surface the reaction force, $R_C$, can be estimated using an equation given by Pesce [4].

$$R_C = mg \frac{EI}{H}$$

where

- $mg$: submerged weight of riser per unit length
- $E$: Elastic modulus
- $I$: second moment of inertia
- $H$: tension in the riser at the TDP

**Large Displacement Dynamic Soil Stiffness**

Large displacement dynamic soil stiffness is used to model the pipe/soil interaction during large riser motions where the pipe breaks away from the soil. Using the equations for soil stiffness and backbone curve an equation for the large displacement dynamic soil stiffness can be written.

$$K = \frac{Q_0}{z_U} = \frac{Q_U}{\Lambda D}$$

where

- $Q_0$: mobilisation distance, calculated using $\Lambda D$
- $Q_U$: a non-dimensional parameter representing the distance, as a function of diameter, that the pipe has to move to mobilise the full soil force.

Observations of SCR trenches during the STRIDE JIP concluded that trench depths, and hence penetration depths are greater than $\frac{1}{2}D$. This indicates that for soil stiffness calculations other than the static stiffness the bearing width can be assumed to be equal to the pipe diameter. This simplifies equation (8) to the following.

$$K = \frac{q_U D}{\Lambda D} = \frac{1}{\Lambda} q_U$$

Substituting the bearing capacity formula, equation (2), into equation (9) and simplifying gives

$$K = \frac{1}{\Lambda} \left( N_c S_U + \gamma z \right)$$

If the trench depth is assumed to be shallow then the $\gamma z$ term is small and equation (10) simplifies to

$$K = \frac{1}{\Lambda} N_c S_U$$

This equation is similar to published equations for the Elastic Modulus of a clay soil that have been calculated using plate bearing tests [5]. This equation has the form.

$$E = \beta S_U$$

where

- $\beta$: a dimensionless parameter determined from experiments, and is 400 for cycling [5] and between 800 and 2500 for static loading on clay soils [6].

The mobilisation distance is the displacement over which the force changes between the backbone curve and 0 and is the displacement of the unloading curve. Observations of the STRIDE and CARISIMA test data show that the normalised mobilisation distance, $\Lambda$, is approximately equal to 0.025 [7]. Using this data equation (11) simplifies to the following.

$$K = 40 N_c S_U$$

The above equation is considered to be conservative as it only models the unloading curve. A less conservative model of large displacement dynamic soil stiffness uses the force between the backbone curve and the soil suction curve with the representative mobilisation displacement. This is illustrated in Figure 5. Using observations of the STRIDE and CARISIMA data [7] this normalised mobilisation distance, $\Lambda$, is found to be 0.1. The force over which this normalised mobilisation displacement is valid is the sum of the backbone curve and the soil suction peak. Conservatively, for short consolidation times, the soil suction force can be assumed to equal the penetration force from the backbone curve. This modifies equation (11) and simplifies to the following.

$$K = \frac{1}{\Lambda} 2 N_c S_U = 20 N_c S_U$$

Further modifications to this model to make it less conservative are possible. These include using the reaction force at the TDP instead of the backbone curve in equation (9), however these modifications and their implications are not explored in this paper.

**Small Displacement Dynamic Soil Stiffness**

The small displacement dynamic pipe/soil interaction model proposed by the STRIDE JIP recommends that vertical downward pipe/soil interaction is modelled using the hyperbolic model developed by Audibert [2]. This model scales the cyclic loading force/displacement curve using the ultimate penetration force at a given depth from the backbone curve and the mobilisation displacement that from the STRIDE and CARISIMA data is 0.025 times the external pipe diameter. The model is generally given in the following form.
\[ Q = \frac{z_D}{A' + B'z_D} \quad (15) \]
\[ A' = \frac{(1 - X)z_U}{Q_U} \quad (16) \]
\[ B' = \frac{X}{Q_U} \quad (17) \]

where
- \( Q \) force per unit length
- \( z_D \) dynamic displacement, the maximum value of which is \( z_U \)
- \( X \) soil parameter which varies between 0.85 (soft clays) to 0.93 (stiff clays)

The hyperbolic soil model can be expanded and rearranged into the following form.

\[ Q = \frac{z_D}{(1 - X)A_D + Xz_D}Q_U \quad (18) \]

For values of dynamic (or secant) soil stiffness, \( K \), the hyperbolic pipe/soil interaction model can be written in terms of bearing pressure, \( q_U \), and takes the following form

\[ K = k_{\text{stiff}}q_U \quad (19) \]
\[ k_{\text{stiff}} = \frac{1}{\Lambda(1 - X) + Xz_D/D} \quad (20) \]

The maximum value of \( k_{\text{stiff}} \), which results in the most conservative value for soil stiffness is at the origin of the hyperbolic pipe/soil interaction model where \( z_D = 0.0 \). This results in the following equation for \( k_{\text{stiff,MAX}} \).

\[ k_{\text{stiff,MAX}} = \frac{1}{\Lambda(1 - X)} \quad (21) \]

Assuming that the seabed is a soft clay soil, \( X = 0.85 \), and that \( \Lambda = 0.025 \) then the maximum value of \( k_{\text{stiff}} \) is 267.

The minimum value of \( k_{\text{stiff}} \) occurs when the dynamic displacement is equal to the mobilization distance, \( z_D = \Lambda D \). This results in the following formula for \( k_{\text{stiff,MIN}} \).

\[ k_{\text{stiff,MIN}} = \frac{1}{\Lambda} \quad (22) \]

Assuming that the normalized mobilization distance, \( \Lambda = 0.025 \) then the minimum value of \( k_{\text{stiff}} \) is 40, which is the same model as the conservative large displacement dynamic soil stiffness model.

Further modifications to this model to make it less conservative are possible. These include applying a reduction factor to the soil stiffness factor to account for soil softening due to continuous cyclic loading.

**Soil Suction**

The soil suction model is based on experimental data observed during STRIDE [9] and CARISIMA [10] testing. These experiments were conducted to look at the effects of pull-out rate, pipe diameter, consolidation time and consolidation load. This model has been used in ABAQUS and ANSYS software and may be implemented in other general purpose FEA programs that have contact elements with a suction modelling capability. The soil suction model has been used to accurately predict the forces on a full scale test riser [11].

For analysis purposes this is modelled in 3 linear phases as shown in Figure 6.

- Suction mobilisation – As the pipe initially moves upwards the suction force increases from zero to the maximum value
- Suction plateau – The suction force remains constant as the pipe moves further upwards
- Suction release – Under further upward movement the suction force reduces from its maximum to zero at the break-out displacement

The soil suction model has two defined limits the maximum uplift resistance force and the break-out displacement from which all points on the soil suction model are derived. The values used to calculate these parameters are defined below and depend on the type of SCR analysis conducted.

**Maximum Soil Suction Force**

Maximum soil suction force is estimated using the formulae below [9].

\[ Q_{S,MAX} = k_c \times k_V \times k_T \times N \times D \times S_U \quad (23) \]
\[ k_V = k_T \left( \frac{V}{D} \right)^{n_F} \quad (24) \]
\[ k_T = k_{TF} \left( \frac{F_C \sqrt{c_T}}{L^2} + C_{TF} \right) \quad (25) \]

where
- \( Q_{S,MAX} \) maximum uplift force per unit length (kN/m)
- \( k_c \) cyclic loading factor (no units)
- \( k_V \) an empirical pull-out velocity factor (no units)
- \( k_T \) consolidation time factor (no units)
- \( k_{TF} \) empirically derived constant from CARISIMA and STRIDE test data (S\( \times \)n\( F \))
- \( V \) pull-out velocity (m/s)
- \( n_F \) empirically derived constant from CARISIMA and STRIDE test data (no units)
- \( k_{TF} \) empirically derived constant from CARISIMA II and STRIDE test data, \( k_{TF} = 0.00033 \) (m\(^2\)/N)
- \( F_C \) consolidation force (N)
- \( c_T \) coefficient of consolidation (m\(^2\)/year)
- \( t \) consolidation time (years)
When using the formula the following assumptions are made:

- The factors \( k_D \), \( n_D \) and \( k_c \) are based on CARISIMA and STRIDE data and may vary with soil type. Recommended values are given in Tables 1 and 2.
- Extreme storm and first order fatigue analysis have a reduced soil suction force due to the following:
  - Short Consolidation Times. The maximum suction force for a pull-out after a short consolidation time can be estimated by using the remoulded undrained shear strength as opposed to the undisturbed undrained shear strength and setting factor \( k_T \) to 1.0. A short consolidation time is analogous to the rest time between dynamic TDP motions.
  - Cyclic Loading. Repeated pull-out in a remoulded trench causes the soil to liquefy and the maximum dynamic soil suction force to drop [9].
- The undrained shear strength to be used is the (undisturbed or remoulded) undrained shear strength at the assumed trench depth
- The effect of pipe peeling is assumed to be negligible

\[
A_B = k_D V D \\
k_D V = k_D X V^n_D \\
k_{DT} = k_{DTF} \frac{F_C \sqrt{C_T t}}{L D^2} + C_{DTF}
\]

Limits of the Soil Suction Model

The empirical parameters used in the soil suction models are based upon the STRIDE and CARISIMA experimental work, and therefore are applicable over the ranges of the experimental test parameters. A summary of the ranges of the experimental parameters, and therefore the soil suction model limits are given in Table 4.

<table>
<thead>
<tr>
<th>Test Parameter</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pull Out Velocity</td>
<td>0.005m/s (0.005V/D)</td>
<td>0.2m/s (0.8V/D)</td>
</tr>
<tr>
<td>Consolidation Time</td>
<td>5min</td>
<td>112Hrs</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>30%</td>
<td>50%</td>
</tr>
</tbody>
</table>

Table 4 – Limits of the Soil Suction Model
Conclusions and Recommendations
The state-of-the-art models presented within this paper have been developed using published data and data from the pipe/soil interaction experiments conducted within the STRIDE and CARISIMA JIP’s. The dynamic soil stiffness models presented are considered to be conservative since they do not account for soil softening due to repeating cycling and use the bearing load as opposed to the TDP reaction force for calculating soil stiffness. Further research is required to reduce the conservatism in the dynamic pipe/soil interaction models and hence reduce the conservatism in predicted SCR fatigue lives.

The soil suction model was developed during the STRIDE JIP and has been used in SCR analysis.

Acknowledgments
The authors are grateful to 2H Offshore Engineering Ltd and BP for their permission and support in publishing this paper.

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STRIDE PHASE IV - CASE STUDIES
% of Rigid Seabed Fatigue Damage of SCR with Varying Linear Seabed Stiffness

![Graph showing the effect of linear soil stiffness on SCR fatigue damage.](image)

Figure 1 – Effect of Linear Soil Stiffness on SCR Fatigue Damage [1]
Figure 2 – Illustration of Pipe/Soil Interaction
Figure 3 – Re-penetration Pipe/Soil Interaction Curves

Figure 4 – Example of Static and Dynamic Soil Stiffness

Figure 5 – Displacements of Unloading and Soil Suction Pipe/Soil Interaction Curves
Figure 6 - Soil Suction Model [9]
When researchers wanted to model steel catenary riser (SCR) behaviour on the seabed, they carried out full scale experiments simulating the bottom end of a deepwater SCR in a harbour area exhibiting a similar mud substrate to that found in the Gulf of Mexico. Christopher Bridge and Neil Willis of 2H Offshore Engineering explain:

Steel catenary risers have become a popular choice for deepwater developments. Hanging from a floating production vessel, they are subject to wave and current loading. Predicting the shape and general forces on a SCR is a relatively simple process, and is used to solve standard catenary equations. Critical to the design however, is the point at which the riser first touches the soil – the so-called touchdown point (TDP).

The understanding of the fluid/riser/soil interaction at this point is limited. It is particularly important as the soil parameters used in SCR analysis can have a significant effect on riser response, especially the predicted fatigue life. If soil parameters and analytical models are chosen too conservatively, they can make the predicted fatigue life unrealistically low. Conversely, using non-conservative soil parameters and soft soil models results in fatigue lives that may be unrealistically high. This prompted 2H to examine closely vertical pipe/soil interaction models for use in SCR analysis.

As the riser moves up and down, the TDP moves along the riser. It has been found that of all the vessel motions, heave causes the greatest stress fluctuations at the TDP. Analysis has shown that a dynamic heave motion of ±1m amplitude can cause the TDP on a riser in 1000m water depth to move horizontally by 10m. The main forms of loading on vessels are:

- First order motions – wave frequency motions caused by wave action on the vessel
- Second order motions – low frequency motions often referred to as slow drift motions
- Static offset – displacement resulting from mean environmental loads such as currents, waves and winds, or system failures, such as failed mooring lines.

In addition to the vessel loads, the current acts directly on the SCR. This causes the riser to flex in the direction of the current, and can invoke high frequency vortex induced vibration (VIV) motions in the riser.

### Seabed

Deepwater oil and gas fields usually have seabeds of soft clay. At the dynamic TDP the riser can bury itself in a deep trench. With increasing distance from the vessel, the SCR gradually rises to the surface where it rests, and effectively becomes a static pipeline.
mechanisms that create these trenches are not clear, however they are thought to be produced by the dynamic motions of the riser combined with sediment transportation effects of the seabed currents.

Storm and current action on a deepwater production vessel can pull the riser upwards from its trench, or laterally against the trench wall. This interaction could cause an increase in the local riser stresses (due to tighter riser curvatures and higher tensions) than those predicted ignoring the seabed trench.

As part of the STRIDE III joint industry project, 2H Offshore Engineering conducted a full-scale test programme to investigate the effects of fluid/riser/soil interaction on catenary riser response and wall stresses at the TDP. The objective was to assess the importance of fluid/riser/soil interaction, and to produce finite element (FE) analysis techniques to predict the measured response.

The test programme was conducted over three months at a harbour location in the west of England. A 110m (360-ft) long 0.1683m (6-5/8 in) diameter, riser was suspended from an actuator on the harbour wall and run out across the seabed to a set of mud anchors. The seabed over this area was flat and undisturbed, and careful probe tests were done to check that there were no hidden obstacles below the mudline.

The harbour test riser was completely instrumented with 13 sets of strain gauges measuring vertical and horizontal bending strain which spanned the TDP area, and load cells measuring the tensions and shear forces at the actuator and the tension at the anchor. In addition, triaxial accelerometers were placed on the actuator. All instrumentation was hard wired back to a real time, 40Hz multi-channel logging station.

From previous core sample tests, the harbour seabed was known to have properties similar to a deepwater Gulf of Mexico seabed. This is made up of soft clay, with an undrained shear strength of 3 to 5 kPa, a sensitivity of 3, a plasticity index of 39%, and a naturally consolidated shear strength gradient below the mudline. The harbour tests were conducted over a six-week period on several test corridors including an open trench, an artificially deepened trench, a backfilled trench and on a rigid platform. For each test corridor a series of tests was conducted to examine the effects of slow drift (pull up and lay down tests) and dynamic motions (day-to-day and second order motions).

The results from the harbour test riser were recorded as bending moment traces versus actuator position at strain gauge locations. Comparisons were made between the bending moment data from a strain gauge during pull up and lay down tests. A negative bending moment corresponds to a sagging bend in the riser.

Initially the pipe location is on the seabed, in the surface zone. As the actuator moves the top of the pipe upwards the pipe location moves into and through the buried zone, until at the end of the actuation the pipe location is free hanging in the catenary zone. An example of the bending moments recorded during a test is shown below. During this actuation the TDP was observed to move 25m towards the anchor.

**OBSERVATIONS OF RISER TRENCHES**

When the harbour test riser was initially placed on the seabed the soil deformed to create a close fitting trench around the pipeline. This close-fitting trench was observed at low tide after the riser had been floated into place.

During the course of the test programme, several trenches were formed. Typically, the trench formed starts where the riser first touches the soil when the actuator is at its lowest position (which it was between most tests). The trench extends towards the anchor and the width increases from 1 diameter to a maximum of 2.5 diameters over a distance of 20m. The trench then...
reduces in width to 1 diameter over the next 40m at which point it is considered to be a static pipeline in the surface zone.

Several candidates were identified for possible trench mechanisms, including:

1. The dynamic motions applied by the actuator, representing the vessel motions, may have dug the trench. However, there was little build up of soil around the top of the trench, which may be expected if the riser had been pushed into the trench walls by the riser motions.

2. The flow of the tides may have scoured and washed away the sediment around the riser, or caused dynamic vibration of the riser. This is not thought to have been the case since the water level at the test area had little significant velocity. Instead the water flow into the harbour was mainly around the harbour walls, and the level rose very gently at the test corridor areas. This was also demonstrated by observing that the empty trenches formed in previous test corridors survived a large number of tides with little change.

Trenches formed during the relatively short test programme, up to 2/3 of the riser diameter deep and up to 2.5 diameters wide. This could occur over a matter of hours, as was shown from cyclic tests where the actuator was left simulating storm waves over a period of time. The presence of water around the pipe and in the trench was found to be fundamental to this trenching mechanism, since tests conducted at low tide without water did not cause the same rate of trench growth.

Observing the storm wave actuation at low tide with the trench artificially water filled showed the effect of the pumping action of the riser in its trench, with high local water velocities around the diameter and along the catenary as it moved up and down. It is believed that this is the major trench forming mechanism, with clouds of thin slurry mud being dispersed away from the trench. As the trench deepens, this slurry may not then escape the trench, but would settle back to cover the pipe and result in the catenary burial seen on a number of offshore riser surveys.